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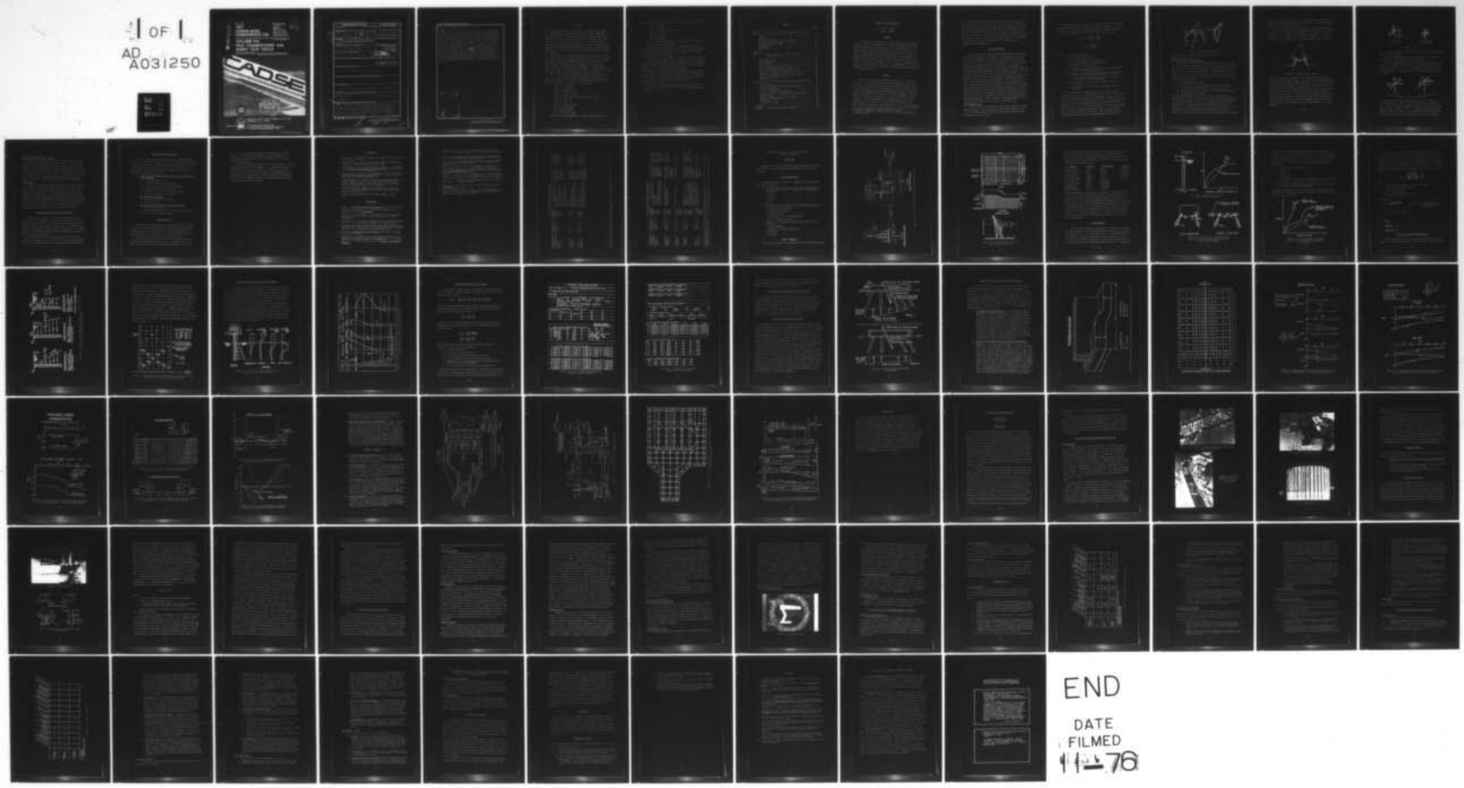

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CORPS-WIDE CONFERENCE ON COMPUTER AIDED DESIGN IN STRUCTURAL ENGINEERING

22-26 September 1975

VOLUME VIII
PILE FOUNDATIONS and SHEET PILE CELLS

Edited by N. RADHAKRISHNAN

Prepared for Office, Chief of Engineers, U. S. Army
Washington, D. C. 20314

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PILE FOUNDATIONS - The state of the art concerning the design of pile foundations is presented. Topics presented include design philosophy, design methods, and the computer programs which are currently available for rigid structures. Selection of the preferred design method is made giving consideration to the types of structures for which they should be used. Recommendations are also made concerning the adoption of a standard computer program for Corps-wide use.

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20. ABSTRACT (Continued).

CELLULAR SHEET PILE STRUCTURES - The first cellular cofferdam, constructed of sheet steel piling, was built in 1909. The cellular sheet pile construction technique is still most commonly used for cofferdams. Other applications are fixed crest dams and weirs, navigation lock walls, mooring cells, retaining walls, and substructures for concrete gravity superstructures. The most common configurations for cellular sheet pile structures are circular cell, diaphragm, and cloverleaf. The circular cell type is preferred for construction in river currents. Diaphragm-type cells are economical for moderate height structures in still water. Where there is no room for stability berms and large cells are needed for stability, the cloverleaf configuration is practical. Three major considerations in the design of these structures are: (a) resistance of sheet piling and connectors to all internally applied loads from the cell fill (including hydrostatic forces), (b) cell stability under all loading conditions, considering all possible modes of failure, and (c) control of seepage quantity, seepage forces, and erosive currents. This paper further discusses the theory and practice of structural design, evaluates available computer programs, and makes recommendations for the development of future programs.

PREFACE

In December 1974, the Automatic Data Processing (ADP) Center, U. S. Army Engineer Waterways Experiment Station (WES), submitted a proposal to conduct a Corps-wide Conference on Computer-Aided Design in Structural Engineering to the Office, Chief of Engineers (OCE). OCE approved the proposal and efforts were started in February 1975 to conduct this Conference. The Conference was conducted in New Orleans, Louisiana, 22-26 September 1975 and was attended by 175 engineers from 48 Corps field offices, OCE, Construction Engineering Research Laboratory, and WES.

This volume contains the papers from Specialty Session D, State-of-the-Corps Art on Pile Foundations and Sheet Pile Cells. Mr. Herman Gray, Chief, Design Branch, ORNED-D, Nashville District, was session chairman and presented a paper. Other papers were presented by Thomas J. Mudd, Structural Engineer, LMSED-DA, St. Louis District; Mr. Billy H. James, Structural Engineer, SWDED-TS, Southwestern Division; and Thurman Gaddie, Chief, Design Branch, Nashville District.

The Conference was successful due to the efforts of a multitude of people. The roles they played were different but they were all directed toward making a concept on "instant dissemination" work. The Organizing Committee for the Conference consisted of:

COL G. H. Hilt, WES
Mr. F. R. Brown, WES
Mr. D. L. Neumann, WES
Mr. J. B. Cheek, Jr., WES
Dr. N. Radhakrishnan, WES--Conference Coordinator
Mr. W. A. Price, WES
Mr. G. S. Hyde, WES
Mr. D. R. Dressler, LMVD
Mr. W. B. Dodd, LMNDE
Ms. E. Smith, LMNDE
Mr. L. H. Manson, LMNDE

An OCE Coordinating Committee also worked enthusiastically to

ensure the success of the Conference. This Committee consisted of:

Mr. C. F. Corns

Mr. R. L. Delyea

Mr. R. F. Malm, OCE Coordinator

Mr. L. G. Guthrie

Mr. D. B. Baldwin

Mr. R. A. McMurrer

The U. S. Army Engineer District, New Orleans, did a remarkable job in playing host to the Conference.

There were 13 Division speakers, 25 moderators, two invited speakers, four technical speakers, and ten session chairmen, who shared the technical load of the Conference. Also, eight computer vendors showed their ware to the participants.

The editor would like to thank all the individuals who served on the committees and the speakers and the moderators for sharing their time and thoughts. Without them the Conference would not have been the success it was. Mr. Dressler, LMVD, and Mr. Price, WES, are specially thanked for their technical guidance and assistance.

This report was edited by Dr. N. Radhakrishnan, Research Civil Engineer, Computer Analysis Branch (CAB) and Special Technical Assistant, ADP Center, under the direct supervision of Mr. J. B. Cheek, Jr., Chief, CAB, ADP Center, and the general supervision of Mr. D. L. Neumann, Chief, ADP Center.

The Director of WES during the Conference and the preparation of this report was COL G. H. Hilt, CE. Mr. F. R. Brown was Technical Director.

CONTENTS

DESIGN OF PILE FOUNDATIONS

by

Billy H. James*

Thomas J. Mudd**

Purpose

The purpose of this paper is to present the state of the art concerning the design of pile foundations. Topics presented include the design philosophy, approximate and more exact design methods, and the computer-aided design techniques used in the Corps of Engineer (CE), other Government agencies, and private industry. Various computer programs are discussed along with the shortcomings of each program. The preferred design methods are selected with consideration given to the complexity of structures designed. Input received from the discussion sessions is reflected in the selections. Recommendations are also given regarding the modification of available programs and their standardization for Corps-wide use.

General

Pile foundation design has advanced greatly since the 1950's in the fields of soil mechanics and structural engineering. Previously, the methods of design were mainly simple analytical or graphical solutions. These have proven to be adequate for the design of minor structures such as bridge abutments, retaining walls, and similar structures. However, these approximate methods are considered inadequate for the design of battered pile foundations for larger, horizontally loaded, structures. Approximate methods are inadequate because of the neglected effects of structure rotations, the lateral resistances of piles, and three-dimensional loads and behaviors. Also, the approximate

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* Structural Engineeer, Southwestern Division (SWD).

** Structural Engineer, St. Louis District, Lower Mississippi Valley Division (LMVD).

design methods provide no check on the deflections and combined axial and bending pile stresses. Hence, the most efficient and economical pile foundation will not be developed. More accurate methods have been developed which perform two- and three-dimensional methematical analyses of the structure-foundation system. Although the mathematics required to overcome the shortcomings of the graphical methods is complex, the analysis is easily solved by the electronic computer. Therefore, the computer has played a significant role in the advancement of pile foundation design techniques.

Design Philosophy

Foundation piles are supporting structural members which transfer loads from the structure to the subsoil. Adequate design will ensure that excessive deflections and stresses in the "structure-pile-soil system" will not occur. The current design practice within the Corps, other government agencies, and outside industry is to analyze the "structure-pile system" and the "soil-pile system" separately, ignoring the nonlinear interaction between the two systems and contact pressures between the structure and soil. The "structure-pile system" is analyzed with applied loads to determine the resisting pile loads. The distribution of the loads to the individual piles is dependent on the amount of vertical and horizontal movements of the structure base. These movements should include both translation and rotational effects. Such movements are a function of (a) structure rigidity, (b) physical property of piles, (c) pile head fixity, and (d) resistance of soil to horizontal and vertical movements of the pile. The various methods used to determine the distribution of loads to the piles are given in subsequent paragraphs. These include the approximate methods, direct stiffness method-rigid base, and finite element techniques.

Approximate methods

Several approximate methods have been used in the past to determine pile loads. These methods ignore the lateral capacity of the piles, three-dimensional effects, pile head fixity, and rotational resistances of the pile group.

The moment of inertia method is used for the design of foundations containing only vertical piles; it is considered acceptable if its limitations are recognized. The applied load may be eccentric about two axes. Individual pile loads are calculated by equations 1 and 2.

$$P_v = \frac{V}{n} \pm V \frac{e_x c_x}{I_y} \pm V \frac{e_y c_y}{I_x} \qquad (1)$$

$$P_h = H \pm \frac{H e_p c_p}{I_p} \qquad (2)$$

where

P_v = axial load on pile

P_h = horizontal load on pile

V = vertical component of applied load

H = horizontal component of applied load

n = number of piles in footing

e_x, c_y = distance from applied load to y-y and x-x axes through center of gravity of pile group

c_x, I_y = moment of inertia of pile group about x-x and y-y axes. Each pile is considered to have a value of unity

e_p = polar distance from horizontal component of resultant to center of gravity of pile group

c_p = polar distance from any pile to center of gravity of pile group

I_p = polar moment of inertia of pile group about center of gravity

For battered pile foundations the Culman graphical method is often used: the method is illustrated in Figure 1. Note that the solution is dependent on the geometry and disregards the lateral pile resistance rotational effects and pile head fixity. Anderson[1] developed the center-of-rotation graphical method which reflects the unequal distribution of pile loads caused by rotation on battered pile foundations. However, assumptions made require the piles to be hinged at cap and have no lateral resistances. These approximate methods should be applied only to simple two-dimensional structures with

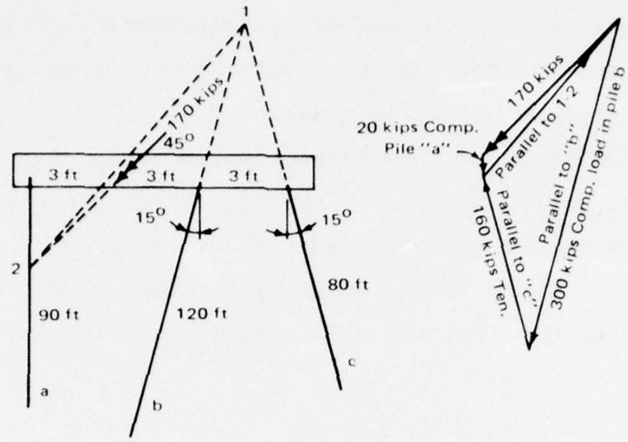


Figure 1. Culman graphical method

lateral loads less than one-fifth the vertical load.[2]

Direct stiffness method-
elastic piles with rigid structure

The graphical methods discussed above consider the piles to have only axial resistances, whereas the piles usually have significant lateral resistances. In 1950, A. Hrennikoff published a workable planar analysis for the design of battered piles. The procedure, which accounts for monolith rotations and lateral pile resistances, stands as a notable contribution to pile foundation design. The analysis is based on the following assumptions:

a. Absolutely rigid pile cap.

b. Each pile load is proportional to pile head displacement.

c. Load-deformation relationships the same for all piles.

d. Problem is restricted to planar or two-dimensional analysis.

e. Footing movements are small.

Aschenbrenner[3] extended the solution from two to three dimensions. Saul[4] later used matrix formulation and refined the Hrennikoff analysis to reflect the lateral, flexural, torsional, and axial stiffnesses of the piles and to permit any position and batter of piles, piles of different sizes, materials, and conditions in the same foundation. Of the numerous methods presented, this method appears to be the most general. The designer should be familiar with matrix methods, structure

soil-pile interactions, and the use of the electronic computer, which is required to perform the computations. This method is an exact numerical analysis for the assumed soil-pile model.

A generalized model of the structure-pile system can be described as a rigid body supported by sets of springs, which represent the actions of the pile forces on the structure when the structure is given unit displacements. An example is the simplified planar model, as shown in Figure 2. It is assumed that the pile head loading for any single pile

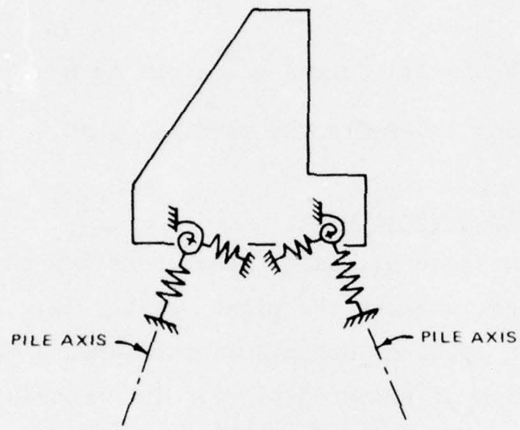
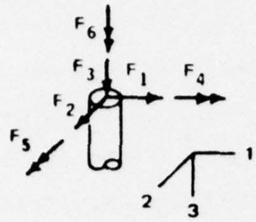


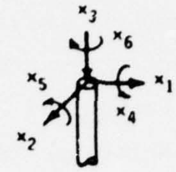
Figure 2. Simplified planar model

in a batter group may be resolved into a combination of axial load, bending moment, shear, and torque. Also, each of these components can be represented by a proper spring constant and results added vectorially to obtain the total movement of the pile head. This method only considers the effect the piles have on the pile cap at the top of the pile. Each pile can therefore be replaced by the proper elastic spring restraints at the pile cap. The forces and displacements along the pile axis are shown in Figure 3, in which axes 1 and 2 are principal axes of inertia and axes 3 coincides with the longitudinal axis of the piling. The pile forces can be equated to the pile displacements, x_i , by the expression

$$F_i = b_i x_i$$



a. Generalized forces



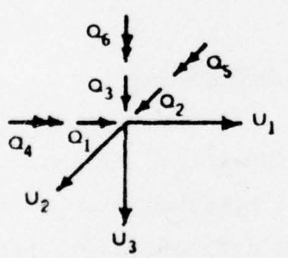
b. Generalized displacements

Figure 3. Piles

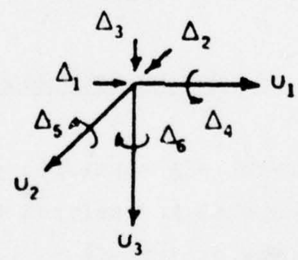
where b_i is the individual pile stiffness influence coefficient called the elastic pile constant.

In a three-dimensional system the pile may be located at a rotated position to the foundation axis and battered.

Once the stiffness matrix is known for the total foundation, the problem is essentially solved; it only requires back substitution to find the allocation of loads to the individual piling. The structure deflections, Figure 4, can be obtained by using the transposed stiffness matrix.



a. Loads



b. Displacements

Figure 4. Structure deflections

Several factors influence the capacity of pile foundations. These factors must be known in order to determine the pile-stiffness matrix. Factors include the subgrade modulus, pile head fixity group and cyclic load reductions, effects of pile driving and jetting procedures, water table variations, and structure-soil contact pressures. A detailed discussion of these parameters is included in Appendix A.

Direct stiffness method-
elastic piles with flexible structure

This method of analysis consists of a finite element structure model with piles represented by a stiffness matrix. A two-dimensional program UFRAME design has been developed by Demsky of the St. Louis District. The Waterways Experiment Station (WES) has developed the SAP IV three-dimensional program to model a flexible structure on piles. These programs are used to reflect structure flexibility in the distribution of pile loads. Sample problems using these programs and comparisons with rigid structure analyses are included in Appendix A.

Other methods

Research studies have been made by Desai, Johnson, and Hargett[5] using a two-dimensional finite element model of the structure-soil-pile system. Parker has also used a two-dimensional model of an elastic structure on nonlinear piles. A research program is underway at the University of Texas at Austin using mathematical models to represent the three-dimensional structure-pile-soil system. The computer program will be used for the design of offshore structures which extend to great water depths. The objective is to obtain the response of the entire system when subjected to wave, wind, and gravity loads.

Design Criteria--Deflections and Stresses

Concurrent with analysis of the "structure-pile" system, the "soil-pile" system is analyzed to verify that computed pile loads and deflections are acceptable. The stresses and deflections in piles resulting from both lateral and axial loads should be investigated and should include applicable cyclic and group loading effects. Allowable design criteria are given in EM 1110-2-2906[6] and AASHTO Standard Specification for Highway Bridges.[7]

The allowable structure deflections should be restricted to 1/4 in. for buildings while 1/4- to 1/2-in. deflections may be acceptable for other structures, depending on the operational requirements.

Available Computer Programs

A survey was made and information obtained on computer programs currently used in the CE. Tables 1 and 2 list two- and three-dimensional programs, respectively. Programs are sometimes used on the local G-225 office computer while other design offices use time share and batch programs on the G-635 computer at WES. Disadvantages of the two-dimensional programs are as follows:

a. 13-C1-C502 Card Program and 41-Z5-002 and 713-F3-A2-150 Time Share Programs.

(1) Assume rigid pile cap.

(2) Structural deflections not computed.

(3) Pile deflections and stresses not computed.

(4) Pile batter restricted to one plane.

(5) Structure base must be horizontal, unstepped.

(6) Pile heads are assumed to be hinged.

b. 713-G1-M413B Card Programs

(1) A Part II program of 713-G1-M413A.

(2) Difficult to use.

c. 713-G-A3840 and 713-F3-A2-210 Time Share Programs

(1) Assume rigid pile cap.

(2) Assume soil modulus variation increasing with depth.

(3) Will not analyze dynamic loads.

Recommendations

Graphical methods are recommended for the two-dimensional design of conventional bridge abutments, small retaining walls, and similar structures, when the applied lateral load on the structure does not exceed one-fifth of the vertical load. For higher lateral-vertical load relationships, more accurate analysis should be used.

Programs 713-Z5-002 and 713-F3-A2-150 are considered to be the preferable two-dimensional programs. It is recommended that one of these programs be modified to eliminate disadvantages (2), (3), (4),

and (6). For structures with stepped or sloping bases, it will be necessary to use the three-dimensional program recommended below. After program modification and revised documentation, the program should be standardized.

Programs 713-F3-A3840 and 713-F3-A2-210 are considered to be the preferable three-dimensional programs. It is recommended that one of these programs be modified to permit the optional use of either a constant, varying, or stepped soil modulus. This program should be standardized for Corps-wide use. Further enhancement should include provisions for the dynamic analysis of pile group foundations subject to earthquake and vibratory loading.

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Table 1

Two-Dimensional Pile Foundation Design Programs

Program Title/ Number	Originating District Date	Equipment/ Language	Program Specifications	Input Data	Output Data
2-D Pile Foundation* Analysis, Hrennikoff/ 13-G1-C502	Savannah District June 1963	G-225 card reader, line printer/ FORTRAN	Rigid pile cap Pile heads fixed or hinged All pile stiffnesses are equal Uses lateral/axial stiffness ratios Pile loads proportional to displacement Structure base horizontal, unstepped Horizontal subgrade modulus is constant	Structure loads, pile and soil properties, pile group pattern dimensions and batter angles	Axial, lateral and moment loads on individual piles as requested
H K Pile (H Pile - Rev program) 41-Z5-002 "Documented"	Vicksburg District 1967	WES G 335 teletype time-share/ FORTRAN	Rigid pile cap Pile heads fixed or hinged All pile stiffnesses are equal Uses lateral/axial stiffnesses ratios Pile loads proportional to displacement Structure base horizontal, unstepped	Structure loads, pile and soil properties, pile group pattern dimensions and batter angles	Axial, lateral and moment loads on individual piles as requested
Hrennikoff Pile Analysis - 2-D/ 713-F3-A2-150 "Documented"	New Orleans District 1968 (Revised 1974)	WES G 635 teletype time-share/ FORTRAN	Rigid pile cap Pile heads fixed or hinged All pile stiffnesses are equal Pile loads proportional to displacement Structure base horizontal, unstepped Horizontal subgrade modulus is constant	Structure loads, pile and soil properties, pile coordinates, batter slope, pile head stiffness, allowable pile loads	Axial and lateral loads on individual piles, overloaded piles and concrete pile stresses

* Rock Island District program 713-G1-F462A and St Paul District program 713-G1-F5160 are identical.

Table 2

Three-Dimensional Pile Foundation Design Programs

Program Title/ Number	Originating District/ Date	Equipment/ Language	Program Specifications	Input Data	Output Data
3-D Pile Foundation Analysis-Phi Batter*/ 713-G1-M413A	Little Rock District Aug 1968 (Revised July 1965)	GE 225 card reader, line printer/ FORTRAN	Rigid pile cap 2-D pile batter Pile head hinged Lateral/axial pile stiffness ratios Pile load proportional to displacement Structure base horizontal, unstepped	Structure loads, pile lateral/axial stiffness ratios, pile-group pattern dimensions and batter angles	Axial, and lateral loads on individual piles as requested (matrix of pile foundation constants) if requested
3-D Pile Foundation Analysis-Beta Batter**/ 713-G1-M413B	Little Rock District Aug 1963 (Revised July 1965)	GE 225 card reader, line printer/ FORTRAN	Supplemental program to 713-G1-M413A Specifications same except: 3-D batter in perpendicular plane is added	Matrix of pile foundation constants from 713-G1-M413A output, 3-D pile-group pattern dimensions and batter angles	Axial and lateral loads on individual piles as requested
3-D Pile Analysis, Hrennikoff/ 713-G1-M4130	Galveston District 1970	GE 225 card reader, line printer/ FORTRAN	Same as 713-G1-M413A above	Same as 713-G1-M413A	Same as 713-G1-M413A
Pile 3-D+/ 713-F3-A3840 "Documented"	St Louis District Sep 1971 (Revised May 1974)	WES G 635 teletype terminal card system WESLIB/ FORTRAN	Rigid pile cap Pile heads fixed, semifixed or hinged Any pile position, batter direction Pile stiffnesses and sizes may vary Pile stiffness, 6 deg of freedom Soil modulus increases linearly Axial stiffness of piles equal to AE/L	Structure loads, pile properties, soil modulus variation (N_H), pile coordinates, batter, head fixity, allowable pile loads and movements	Structure movements, loads and movements of individual piles as requested, list of overstressed piles
3-D Analysis of Pile Foundations/ 713-F3-A2-210	New Orleans District Jan 1970 (Revised Mar 1975)	WES G 635 time-share or remote batch terminal/ FORTRAN	Same as program 713-F3-A3840 Except either constant or linearly Varying soil modulus	Same as program 713-F3-A3840 except constant or variable modulus permitted	Same as program 713-F3-A3840

* Program used by Nashville, modified for fixed or hinged pile head by Mobile District.

** Program used by Galveston District.

† Pittsburgh District program 713-H4-370, Memphis District program "New Pile" and Vicksburg District program 713-F3-A3840 are identical.

Appendix A: Discussion Analysis and Design of Pile-Group Foundations

Given Data

Assume the structure geometry and stability analysis have been determined prior to performing the pile-group analysis and design (Figure A1).

Preliminary Design

Determine the types of piles, pile capacities, and preliminary pile layout (Figure A2).

a. Determine types of pile based on economic studies, availability of materials, nature of loading, and site foundation environment.

- Steel H
- Steel pipe
- Concrete
- Wood

b. Determine pile load carrying capacities and end conditions.

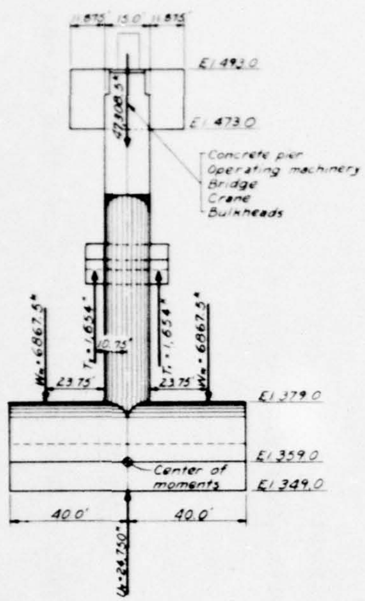
- Allowable axial compression
- Allowable axial tension
- Allowable bending
- End bearing or friction or combination
- Fixed or pinned to structure

c. Determine the preliminary pile layout and batters (if required) by:

- Graphical methods
- Comparison with other similar jobs
- Experience
- Intuition

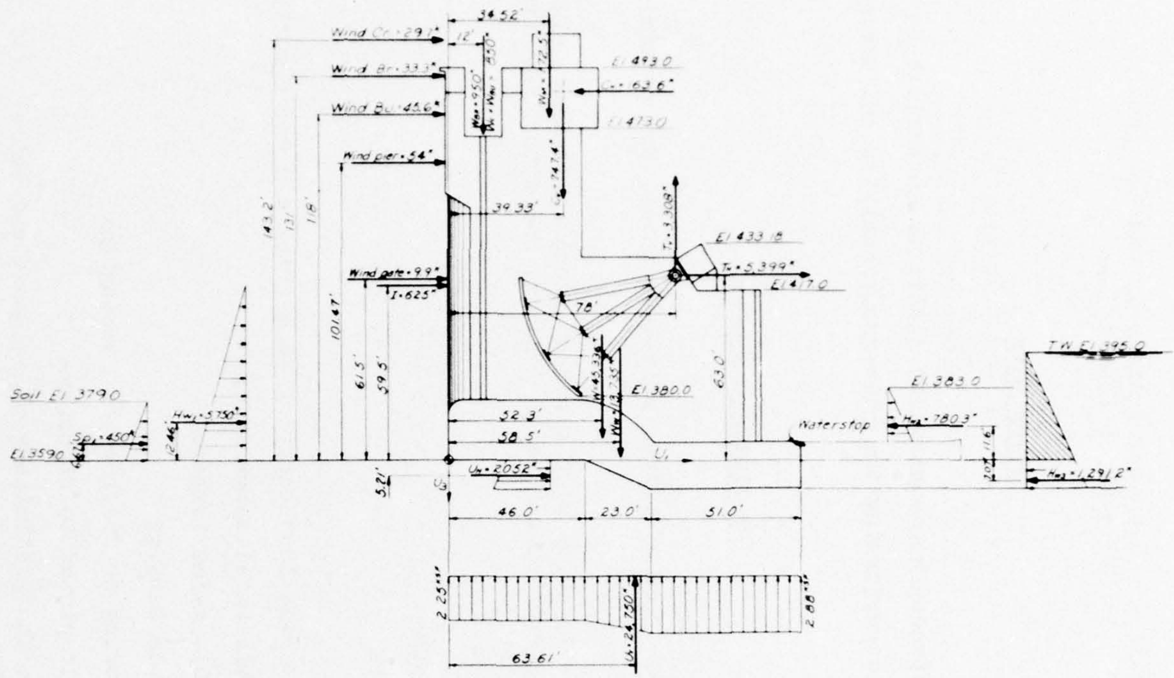
Type of Analysis

Once these preliminaries have been dealt with, then we can perform



CASE 3A (III)

TRANSVERSE LOADING DIAGRAM



CASE 3A (III)

NORMAL LOADING DIAGRAM

Figure A1. Loading condition for pile-founded navigation dam pier

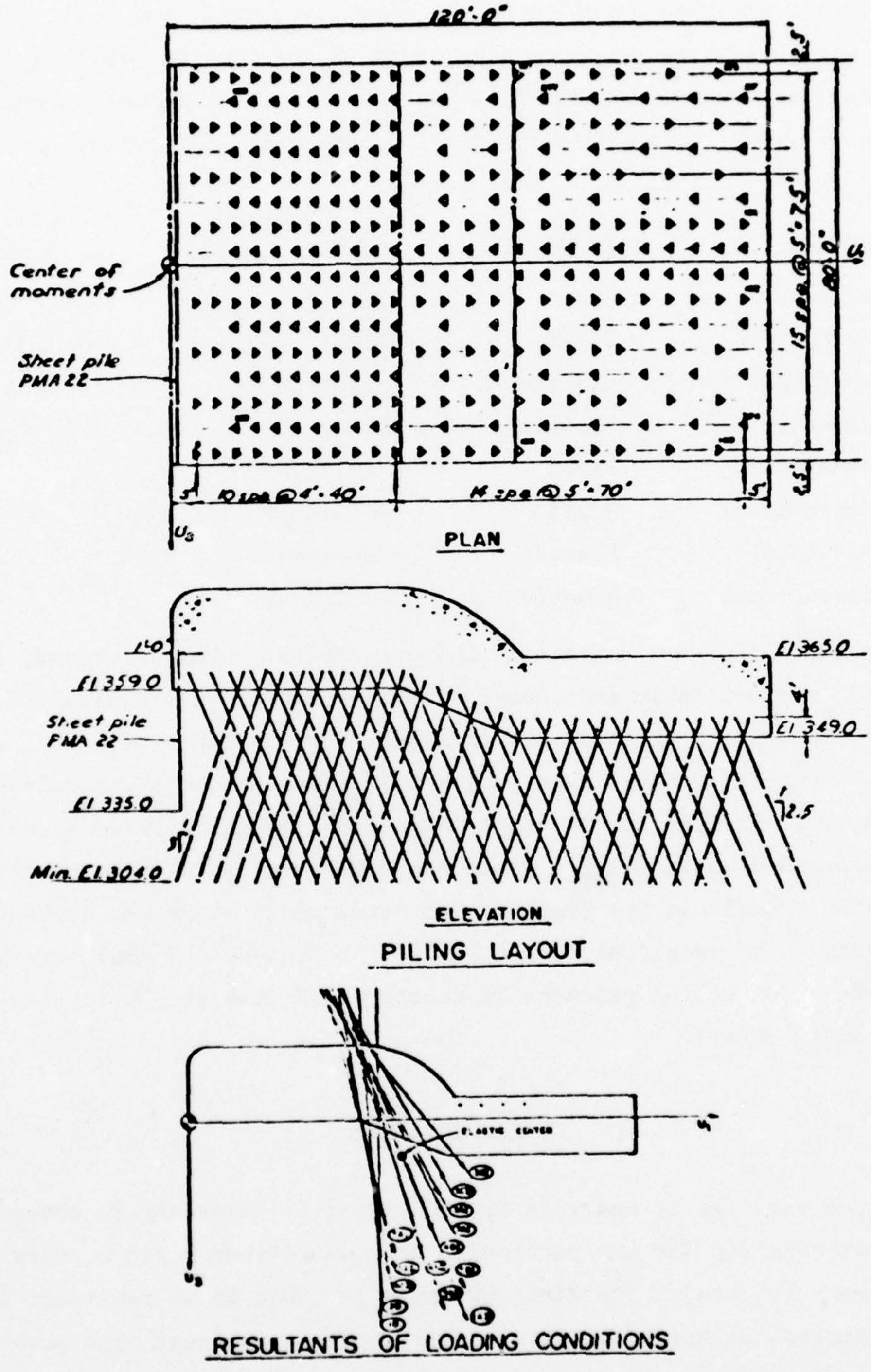


Figure A2. Pile-group layout for pile-founded navigation dam pier

some type of analysis considering the structure, pile, and soil interaction flexibilities, based on the nature of the applied loadings (vertical, horizontal overturning, two-dimensional or three-dimensional loadings). Thus it becomes necessary to choose a tool for the analysis from among the following options:

Loading	Structure	Pile Constants	Program
Two-dimensional	Rigid	Elastic	713F3A3840
Three-dimensional	Rigid	Elastic	713F3A3840
Two-dimensional	Elastic	Elastic	713F3A3910
Three-dimensional	Elastic	Elastic	SAP PILE
Two-dimensional	Rigid	Nonlinear	
Three-dimensional	Rigid	Nonlinear	
Two-dimensional	Elastic	Nonlinear	
Three-dimensional	Elastic	Nonlinear	

The above programs are contained in the handouts. Other programs, such as HPILE, are available from other Districts and provide equivalent analysis as do some of the above programs; however, they will not be discussed here. The available programs that can handle practical design problems will consider up to three-dimensional loadings on an elastic structure and elastic piles. Nonlinear aspects of soil-pile-structure interaction remain in the research and development stage and are not considered to be practical design tools. At present the nonlinear aspects are under active research by groups at WES and the University of Texas, among others.

Pile Constants

Once the type of analysis is chosen, it is necessary to determine the input required for the particular program (either rigid or flexible structure solutions). The first of these is: how do we represent the pile response, or how do we determine the pile constants? The pile constants can be defined as equivalent springs which simulate the pile-soil interaction behavior under load (Figures A3 and A4). The pile can

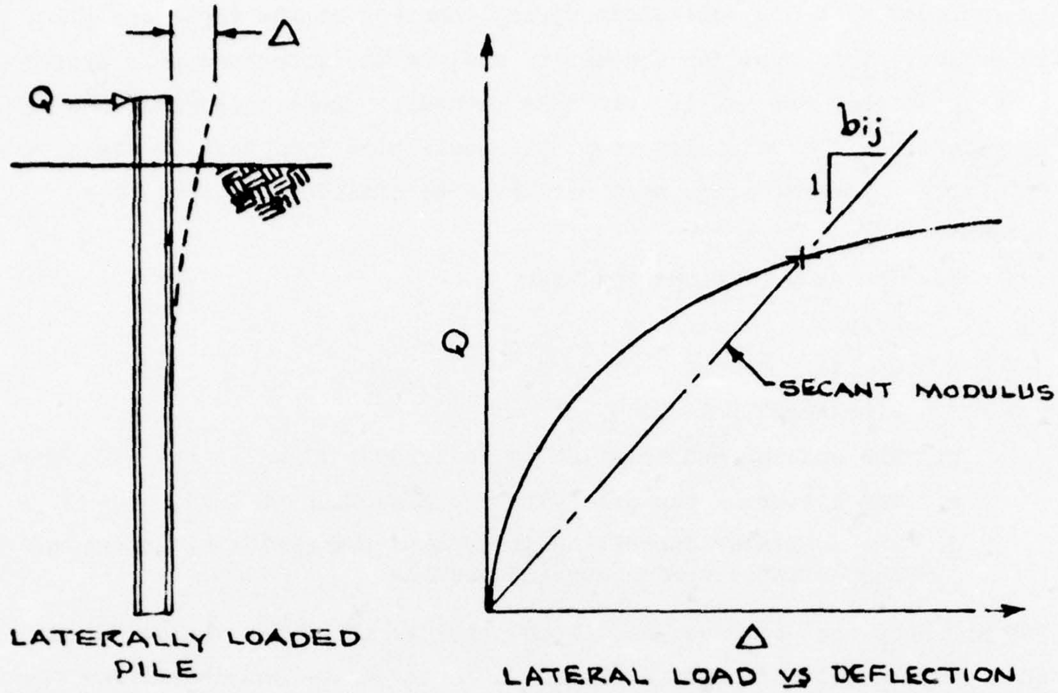


Figure A3. Load versus deflection for a single laterally loaded pile

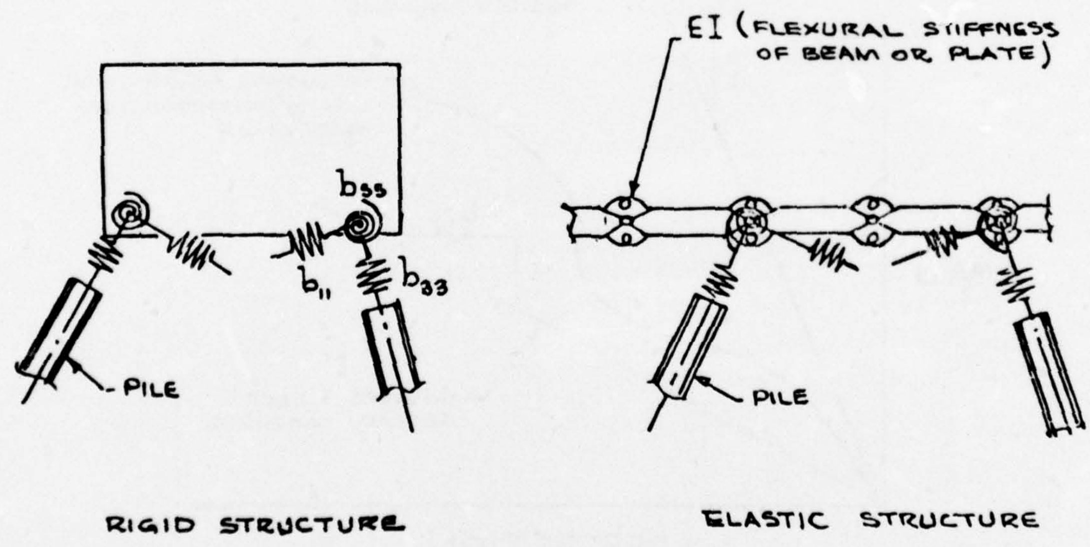


Figure A4. Idealization of elastic pile and structure response by replacement with equivalent elastic springs

be replaced with its equivalent spring constant at the structure pile interface. This provides a means to analyze the structure pile system by using structural matrix stiffness methods. These pile constants can be determined theoretically from full scale pile load test or again by intuition. Several other factors should be considered during this determination:

a. The nature of the loading:

   Static

   Cyclic

   Vibratory

b. The spacing and location of individual piles in the pile group.

c. The batter of the pile with the direction of load.

d. The nonlinear aspects as it affects the choice of an equivalent elastic spring (secant modulus).

One approach that we have used is to consider all these variables and then perform a limit analysis based on conservative assumption for the variable effects (Figure A5).

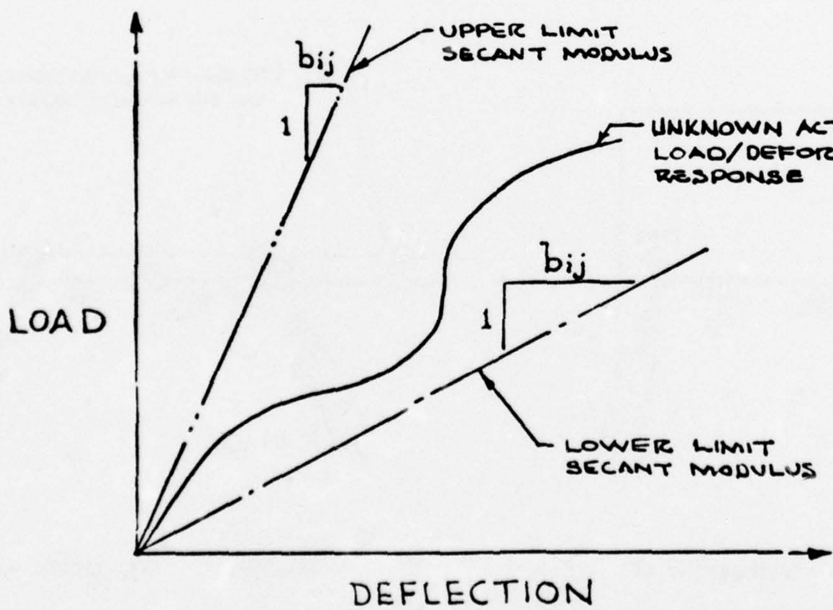


Figure A5. Limit assumptions for unknown pile-soil interaction behavior under applied load

This approach relies heavily on the judgment and experience of the structural or foundation engineer to determine reasonable conservative values for the limit design. However, as it turns out, the results of the analysis are not very sensitive to those assumed values. For instance, Bowles believes that the structure influences the soil behavior much more than the soil influences the structure behavior. A well-known equation to determine a modulus of subgrade reaction for footing behavior is:

$$K_s' = (0.65) \left(\sqrt[12]{\frac{E_s B^4}{E_b I_b}} \right) \left(\frac{E_s}{1 - \mu^2} \right)$$

where

K_s' = modulus of subgrade reaction (soil spring)

E_s = soil stress-strain modulus

B = width of footing

$E_b I_b$ = flexural rigidity of footing

μ = Poisson's ratio

Thus the factor

$0.65 \sqrt[12]{\text{Any number}}$ 0.8 to 1.7 for practical ratios of footing rigidity versus soil

and

$$K_s' = (0.8 \text{ to } 1.7) \frac{E_s}{1 - \mu^2}$$

Try

$$\sqrt[12]{10} = 1.2$$

$$\sqrt[12]{10,000} = 1.8$$

$$\sqrt[12]{100,000} = 2.6$$

Soil-Pile Interaction Consideration

Assumptions which can be used to determine the lateral pile constants (b_{11}) from a theoretical basis are as shown in Figure A6. These

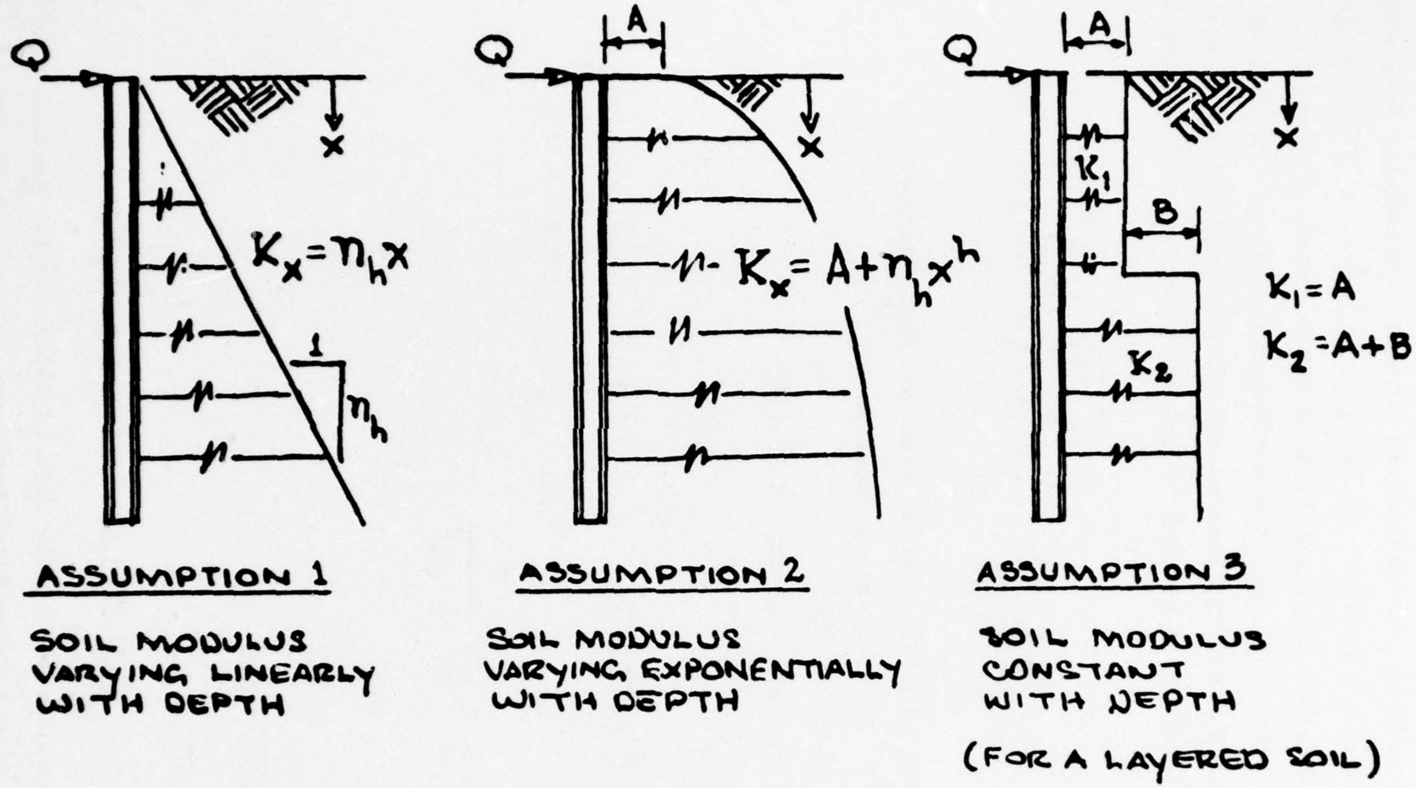


Figure A6. Assumptions for coefficient of horizontal subgrade modulus for various assumed soil conditions

assumptions are based on the Winkler approach first proposed by E. Winkler in about 1867. These assumptions assume that the soil mass can be treated as a series of independent springs which support the structural pile. Other formulations have been based on treating the soil mass as an elastic continuum (FEM approach). Similar assumptions can be used to develop the axial (b_{33}) and rotational (b_{55}) spring pile constants. Programs 713-F3-A3840 (rigid base pile analysis) and SAP PILE (flexible base analysis) calculate the pile constant stiffness matrix internally based on assumption 1 (subgrade modulus that varies linearly with depth). In addition, program A713-G-A3840 has the capability to input a previously defined pile constant stiffness matrix that may have been calculated using other assumptions. The pile constant stiffeners matrix calculated in the above two programs takes the form shown in Figure A7.

$$[b_i] = \begin{bmatrix} b_{11} & 0 & 0 & 0 & b_{15} & 0 \\ 0 & b_{22} & 0 & b_{24} & 0 & 0 \\ 0 & 0 & b_{33} & 0 & 0 & 0 \\ 0 & b_{42} & 0 & b_{44} & 0 & 0 \\ b_{51} & 0 & 0 & 0 & b_{55} & 0 \\ 0 & 0 & 0 & 0 & 0 & b_{66} \end{bmatrix}$$

$$= \begin{bmatrix} K_1\frac{EI_x}{T_x^3} & 0 & 0 & 0 & -K_5\frac{EI_x}{T_x^2} & 0 \\ 0 & K_1\frac{EI_y}{T_y^3} & 0 & K_6\frac{EI_y}{T_y^2} & 0 & 0 \\ 0 & 0 & K_2\frac{AE}{L} & 0 & 0 & 0 \\ 0 & -K_6\frac{EI_y}{T_y^2} & 0 & K_3\frac{EI_y}{T_y} & 0 & 0 \\ K_6\frac{EI_x}{I_x^2} & 0 & 0 & 0 & K_3\frac{EI_x}{T_x} & 0 \\ 0 & 0 & 0 & 0 & 0 & K_4\frac{JG}{L} \end{bmatrix}$$

FIXITY	100%	50%	0%
K_1	1.075	.616	.411
K_2	1	—	—
K_3	1.5	.7845	0.
K_4	2	—	—
K_5	1.0	.499	0.
K_6	1.0	.308	0.

1. 1 OR 1/2 (END BEARING OR FRICTION ASSUMED BY SOME DESIGNERS)
2. TORSION (ASSUMED ZERO BY SOME DESIGNERS)

WHERE:

$$T = \sqrt[5]{\frac{EI}{n_h}}$$

$$K_x = n_h x$$

N_h Can be computed from load tests on piles for assumption 1 by $N_h = \frac{CQ^{5/3}}{y5/3(EI)2/3}$

Figure A7. Stiffness matrix for a single pile for a horizontal subgrade modulus that varies linearly with depth

Matlocks Recursive Solution for Beam Columns

A program that has been very helpful in determining pile constants when a theoretical solution is not available is Matlocks Recursive Solution. This program, 713F3A350, is contained in the handouts. The program solution, based on finite difference techniques, can model general soil-pile interaction conditions. It is possible to develop pile head load response for lateral and rotational deformations. For instance, the following condition can be modeled for soil support starting some distance below the top of the pile (Figure A8). If deflection, rotation, shear, and moment diagrams are desired for the length of individual piles in a pile group, then this program can be used for these calculations. The load pile head deformation results from this program solution can be used as the elements in the pile constant (b) matrix input to the rigid base solution (Figure A9).

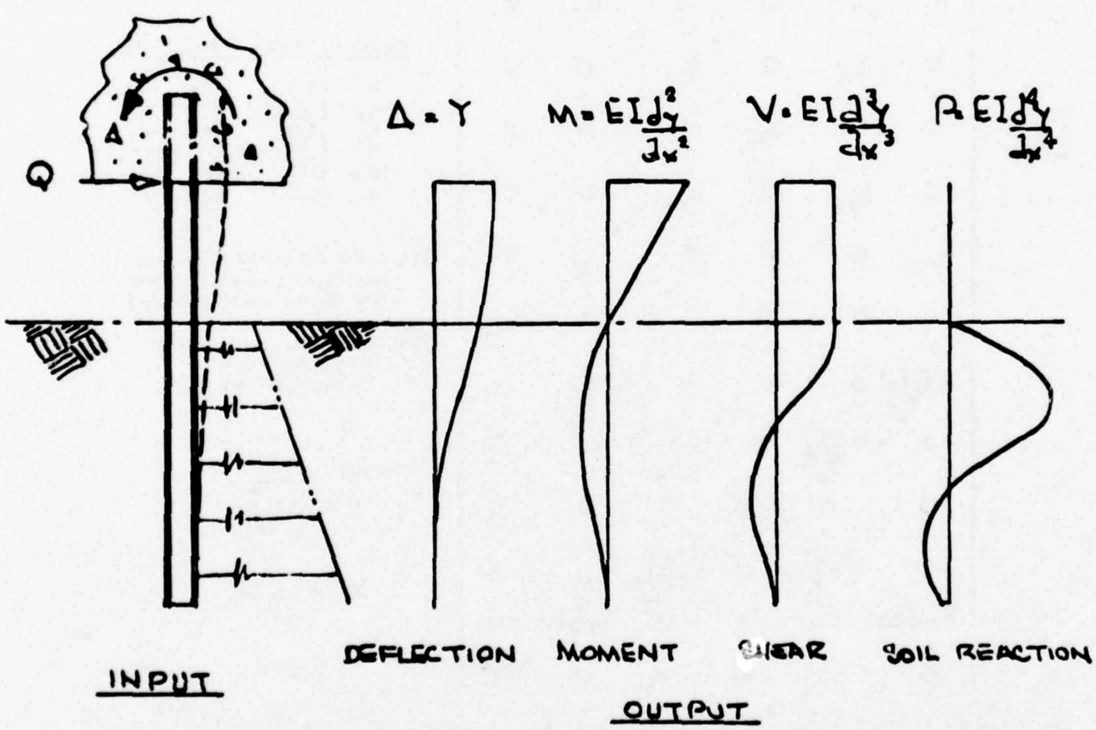


Figure A8. Theoretical load, deflection, moment, shear, and soil reaction curves

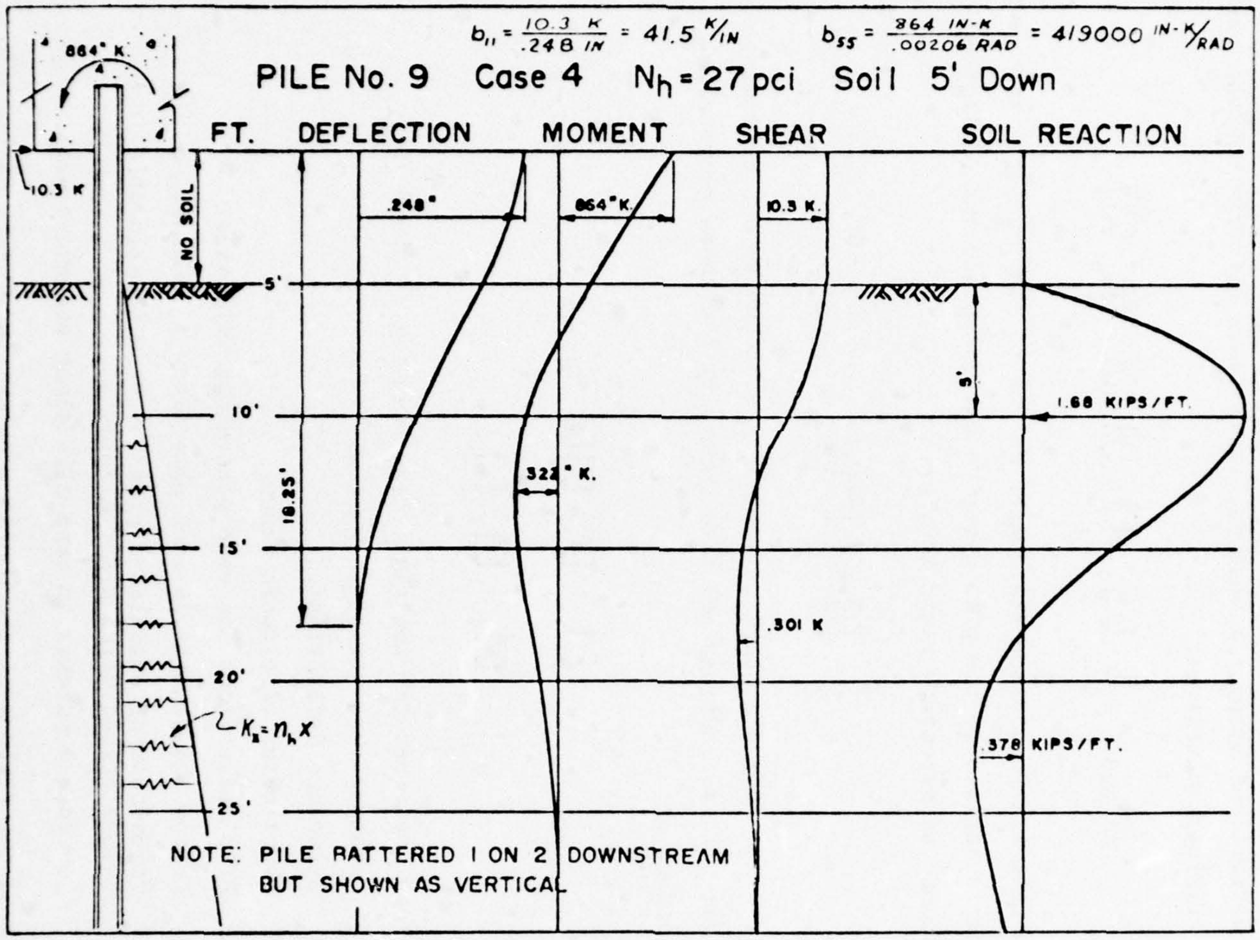


Figure A9. Load, deflection, moment, shear and soil reaction curves developed from a Matlock recursive solution for real applied loads

Rigid Base Solution for Pile Groups

This program is based on Saul's formulation for solving the foundation deflections by matrix methods, which takes the form for the foundation stiffness matrix of:

$$[S]_{6x6} = \sum_{i=1}^{n} [s']_i = [C]_i [a]_i [b]_i [a]_i^T [C]_i^T$$

By inverting the foundation stiffeners matrix [S] and multiplying the load matrix applied to the pile group we can calculate the deflected position of the structure rigid body or:

$$[\Delta] = [S]^{-1} [Q]$$

Once the rigid body structure deflections are known we can determine individual pile deflections and pile forces by the known geometrical relationships or:

$$[X]_i = [a]_i^T [C][\Delta]$$

$$[F]_i = [b]_i [X]_i$$

Sample input required for the rigid base solution is shown in Figure A9. Output consists of (Figure A11):

a. Structure deflections.

b. Coordinates of elastic center.

c. Pile deflections for each pile along pile axis.

d. Forces and moments in each pile along pile axis.

e. Comparison of calculated pile forces with allowable forces (overstressed piles are flagged).

f. Forces and moments in each pile along structure axis.

A useful output feature is the coordinates of the elastic center of the pile group. If the resultant loads and the elastic center are plotted, then a feel for the efficiency of the pile group can be obtained. If

STRUCTURE - RIGID BASE SOLUTION

NO. OF PILES = 77   B MATRIX IS CALCULATED FOR EACH PILE

1. TABLE OF PILE AND SOIL DATA

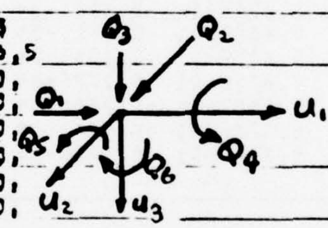
PILE NOS.

```
 1  16   E = 0.42E 07     IX = 0.3540E-01    IY = 0.1260E-01
         AREA =     0.1   LENGTH =  100.0    NH =    6.912
         K1=0.411  K2=1.000  K3=0.       K4=0.        K5=0.        K6=0.
         NO OF ROWS = 7.0
         ALLOWABLES =  FA= 354.8  FB4=5920.0  FB5=8610.0
         CALOW= 200.0  TALOW=  80.0
```

THE B MATRIX FOR PILES 1 THRU 16 IS

0.153E 03	0.	0.	0.	0.	0.
0.	0.101E 03	0.	0.	0.	0.
0.	0.	0.623E 04	0.	0.	0.
0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.

GLOBAL AXIS
RIGHT HAND RULE



2. TABLE OF PILE COORDINATES AND BATTER

PILE NO.	BATTER	ANGLE	U1	U2	U3
1	VERTICAL	0.	4.0	0.	0.5
2	VERTICAL	0.	8.5	0.	0.
3	VERTICAL	0.	13.0	0.	0.
4	VERTICAL	0.	17.5	0.	0.
5	VERTICAL	0.	22.0	0.	0.
6	2.00	0.	32.5	0.	0.
7	2.00	0.	37.5	0.	0.

3. STIFFNESS MATRIX S FOR THE STRUCTURE

0.242E 09	-0.126E-03	0.173E 06	0.976E 03	-0.979E 07	-0.108E 04
-0.126E-03	0.225E 09	0.241E-03	-0.219E 05	-0.140E-01	0.106E 07
0.173E 06	0.241E-03	0.118E 07	0.612E 04	-0.534E 08	-0.976E 03
0.976E 03	-0.219E 05	0.612E 04	0.169E 09	-0.269E 06	-0.303E 08
-0.979E 07	-0.140E-01	-0.534E 08	-0.269E 06	0.310E 10	0.530E 05
-0.108E 04	0.106E 07	-0.976E 03	-0.303E 08	0.530E 05	0.960E 08

3A FLEXIBILITY MATRIX F FOR THE STRUCTURE

0.474E-07	-0.494E-08	-0.770E-07	0.153E-10	0.136E-07	0.105E-09
-0.494E-08	0.991E-04	0.918E-09	-0.195E-06	0.309E-11	-0.116E-05
-0.770E-07	0.918E-09	0.390E-05	-0.375E-10	0.669E-07	-0.202E-10
0.153E-10	-0.195E-06	-0.375E-10	0.666E-08	-0.920E-13	0.427E-08
0.136E-07	0.309E-11	0.669E-07	-0.920E-13	0.152E-08	-0.672E-13
0.105E-09	-0.116E-05	-0.202E-10	0.427E-08	-0.672E-13	0.246E-07

Figure A10. Input to 3-dimensional pile-group analysis program 713F3A3840

COORDINATES OF ELASTIC CENTER IN PLANE 1-3
EC1 = 44.078 EC3 = -8.976

COORDINATES OF ELASTIC CENTER IN PLANE 2-3
EC2 = 0.005 EC3 = 0.973

COORDINATES OF ELASTIC CENTER IN PLANE 1-2
EC1 = 47.277 EC2 = 0.004

********* LOADING CONDITION NO. 1*********

4. MATRIX OF APPLIED LOADS Q

Q1	Q2	Q3	Q4	Q5	Q6
10267.0	0.	6864.0	0.	-529367.0	0.

5. STRUCTURE DEFLECTIONS

D1	D2	D3	D4	D5	D6
0.409E-01	-0.460E-04	-0.945E-02	-0.517E-07	-0.204E-03	0.972E-06
.49 in		.113 in		.0002449	

6. PILE DEFLECTIONS ALONG PILE AXIS

PILE	X1	X2	X3	X4	X5	X6
1	0.396E-01	-0.418E-04	-0.863E-02	-0.517E-07	-0.204E-03	0.972E-06
2	0.409E-01	-0.377E-04	-0.771E-02	-0.517E-07	-0.204E-03	0.972E-06
3	0.409E-01	-0.334E-04	-0.679E-02	-0.517E-07	-0.204E-03	0.972E-06
4	0.409E-01	-0.290E-04	-0.587E-02	-0.517E-07	-0.204E-03	0.972E-06
5	0.409E-01	-0.246E-04	-0.495E-02	-0.517E-07	-0.204E-03	0.972E-06
6	0.378E-01	-0.144E-04	0.158E-01	-0.481E-06	-0.204E-03	0.846E-06
7	0.374E-01	-0.959E-05	0.167E-01	-0.481E-06	-0.204E-03	0.846E-06

7. PILE FORCES ALONG PILE AXIS

PILE	F1	F2	F3	F4	F5	F6	CBFTR
1	6.1	-0.0	-53.8	0.	0.	0.	0.15
2	6.3	-0.0	-48.1	0.	0.	0.	0.14
3	6.3	-0.0	-42.4	0.	0.	0.	0.12
4	6.3	-0.0	-36.6	0.	0.	0.	0.10
5	6.3	-0.0	-30.9	0.	0.	0.	0.09
6	5.8	-0.0	98.4	0.	0.	0.	0.28
7	5.7	-0.0	104.1	0.	0.	0.	0.29
8	5.7	-0.0	109.8	0.	0.	0.	0.31
9	5.6	0.0	115.5	0.	0.	0.	0.33

8. PILE FORCES ALONG STRUCTURE AXIS

PILE	F1	F2	F3	F4	F5	F6
1	6.1	-0.0	-53.8	0.	0.	0.
2	6.3	-0.0	-48.1	0.	0.	0.
3	6.3	-0.0	-42.4	0.	0.	0.

Figure A11. Output from 3-D pile-group analysis program 713F3A3840

the eccentricity between the line of action of the resultant of the loads and the elastic center is minimized then overturning is minimized making efficient use of all the piles (Figure A12).

Calculation of Shears and Moments in Pile Cap

To further enhance this program, the output pile forces from the rigid base solution can be used as input to another program (Slab Shears and Moments Program 713F3A3900) along with other applied dead and live loads to develop the shear and moment diagrams for selected two-dimensional strips through the structure. Shear continuity between adjacent strips should be considered as an applied load to satisfy statics. This program is contained in the handouts.

Flexible Base Solution (SAP PILE)

Recently SLD funded WES to include a pile constant element (defined with assumption 1) into the SAP4 library of finite elements. This work has just been completed by Wayne Jones and Bill Boyt of the Computer Analysis Branch (CAB) at WES. Single and double precision versions of this routine are available. The pile constant element can be used with any of the other SAP library elements considering compatibility of the degrees of freedom, i.e., beam column, plate, brick, etc., to model elastic structures founded on elastic pile groups. We have tested this pile element with the beam column and plate elements and this appears to provide a powerful analysis tool for our structural design of structures founded on pile groups that are less than rigid. A comparison of the results from the analysis of some classes of hydraulic structures reveals that there may be significant differences in the calculated pile forces using the different assumptions of rigid versus flexible structures. Results from the rigid is not conservative in many cases. A draft documentation for this pile element is contained in the handouts. Use of the flexible base solution and a comparison with rigid base solutions are illustrated in the following examples.

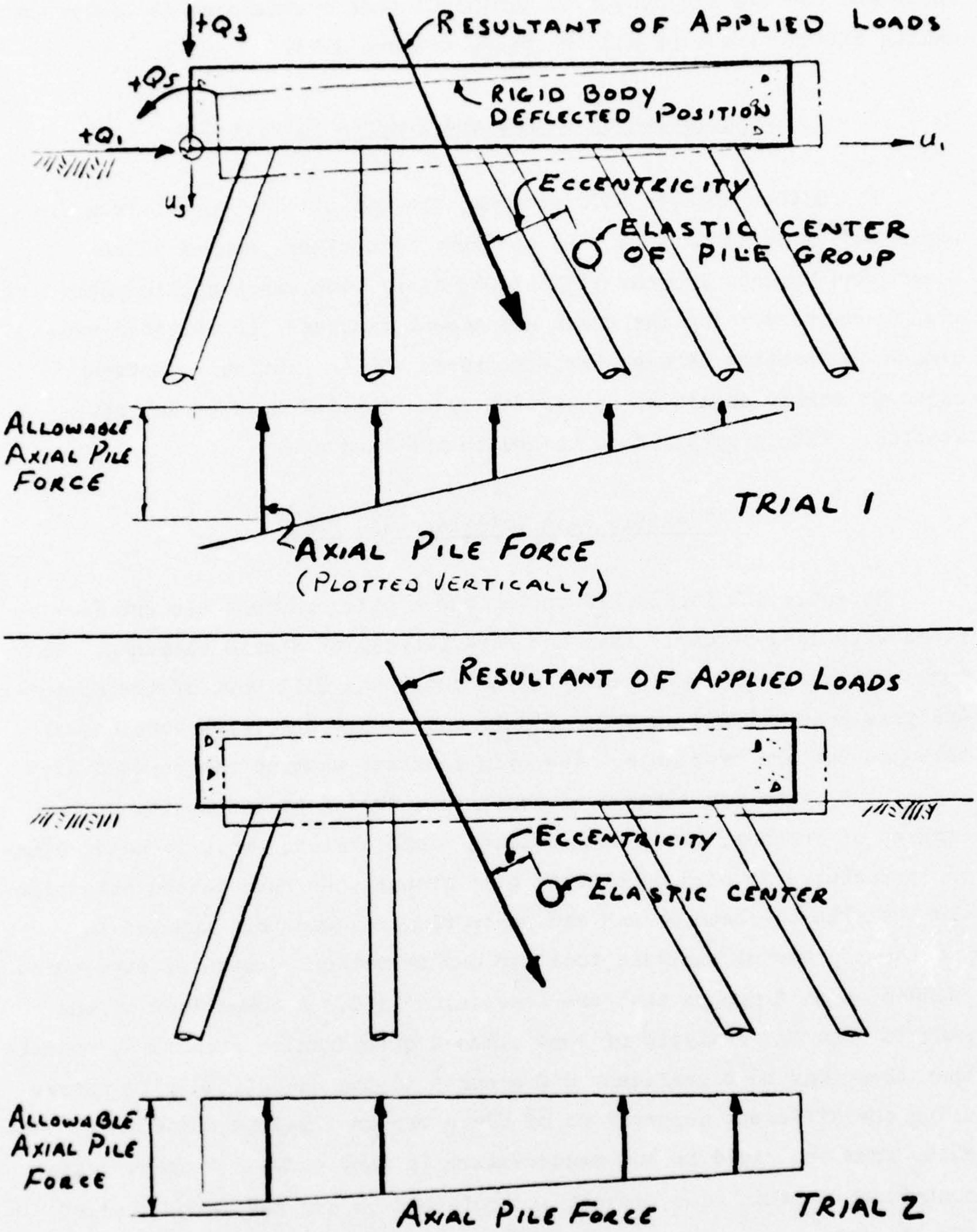


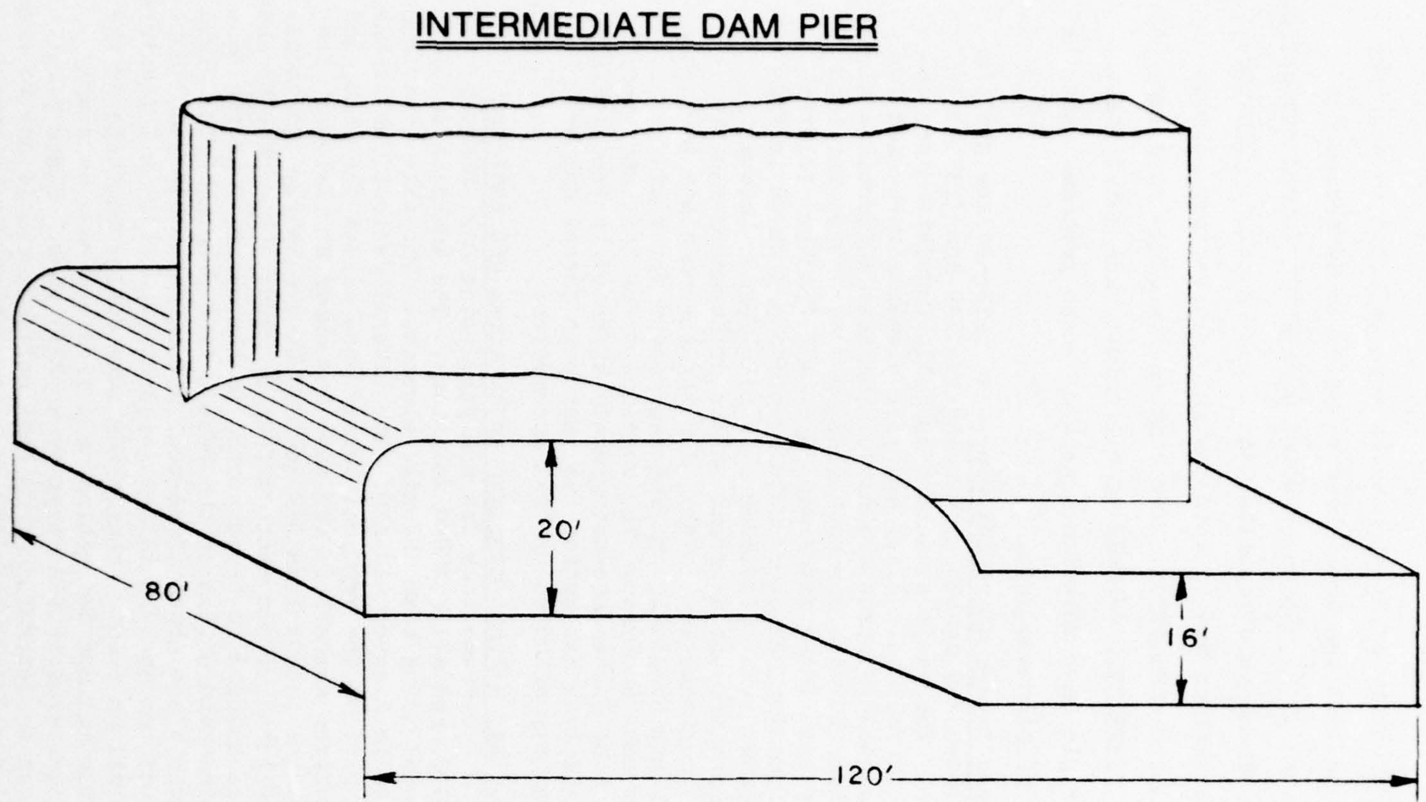
Figure A12. Elastic center and efficiency of pile groups

Examples of Rigid Versus Flexible Base Analysis

Several different types of structures were selected to test the SAP PILE solution against the rigid base solution to determine the sensitivity of the results. Those selected were a "T" shaped navigation dam pier, a lock U-Frame gate monolith, and a pump station. The purpose of this investigation was to gain insight into various methods of analyzing pile-founded structures. The computer programs used were SAP PILE, STRUDL, and 713F3A3910 (UFRAME) for flexible base solutions and 713F3A3840 for a rigid base solution. Each of these programs uses the Hrennikoff method of pile analysis.

a. "T" shaped intermediate dam pier - An intermediate dam pier was analyzed 3-dimensionally using the SAP and 713F3A3840 programs. The basic structure and pile foundation are described as follows. Plate and pile elements were used for this analysis (Figures A13-A14). The base deflections along and across the base for each load case are shown for both flexible base (SAP) and rigid base (A713F3A3840) solutions. Pile forces for selected cross sections are shown in Figures A15-A17. The differences in pile forces between assumptions are not as great as the differences for the U-Frame lock analyses. Thus, for the intermediate pier, the base moments should be in close agreement for rigid and flexible base analyses. The conclusion for this structure is that base forces are not greatly affected by the flexible base/rigid base assumption but that pile forces can be significantly affected by that assumption.

b. Lock gate bay U-Frame monolith on vertical and battered H-piles - A U-Frame lock was analyzed using SAP, STRUDL, 713F3A3910, and A713F3A3840 solutions. The analysis was 2-dimensional using beam and pile elements. The pile locations are shown in Figures A18-A19. The lateral deflections of the lock, a plot of the vertical deflections across the base, and the resulting moments in the base are shown as follows: The base moments output from SAP and STRUDL are nearly identical. UFRAME (713F3A3910) moments differ by about 10 percent because piles are assumed to be connected to the centroid of the base rather than the bottom of the base by the program. The rigid base moments are about 15 percent greater than SAP and STRUDL moments at the maximum values (Figures A20-A21). All flexible base solutions predict nearly the same deflected shape of the base. The maximum deflection of a flexible base is nearly 100 percent greater than that of a rigid base. Since pile forces can be determined from base deflections, it can be seen that the flexible base/rigid base assumption significantly



E_c = 432,000 KSF
ν = .2
γ = .150 KCF

Ep = 4176000 KSF
Nh = 51.84 KCF
HP 14 x 73

Figure A13. Analysis of a pile-founded navigation dam pier assuming rigid and flexible bases

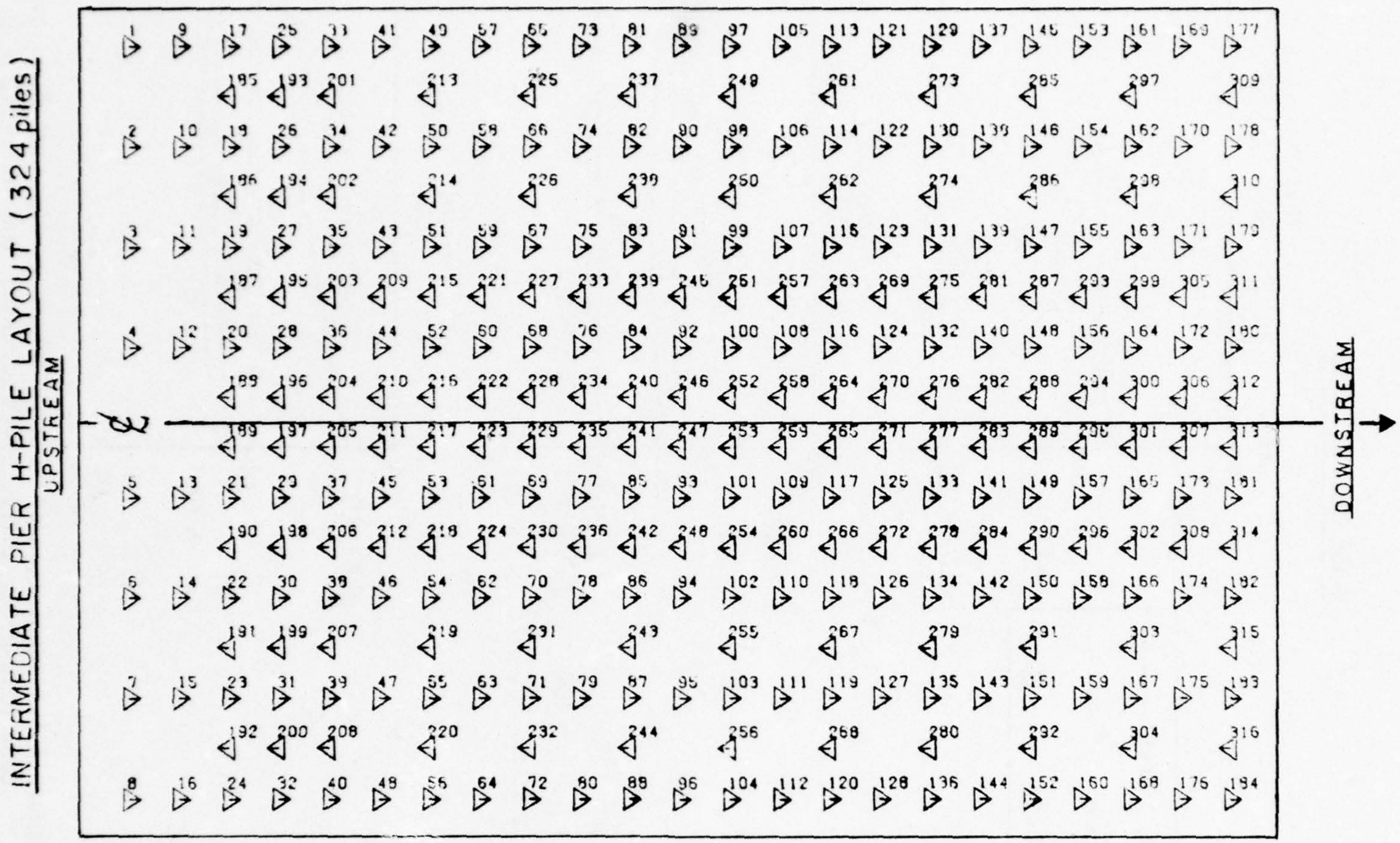


Figure A14. Pile layout used for rigid and flexible base pile-group analysis

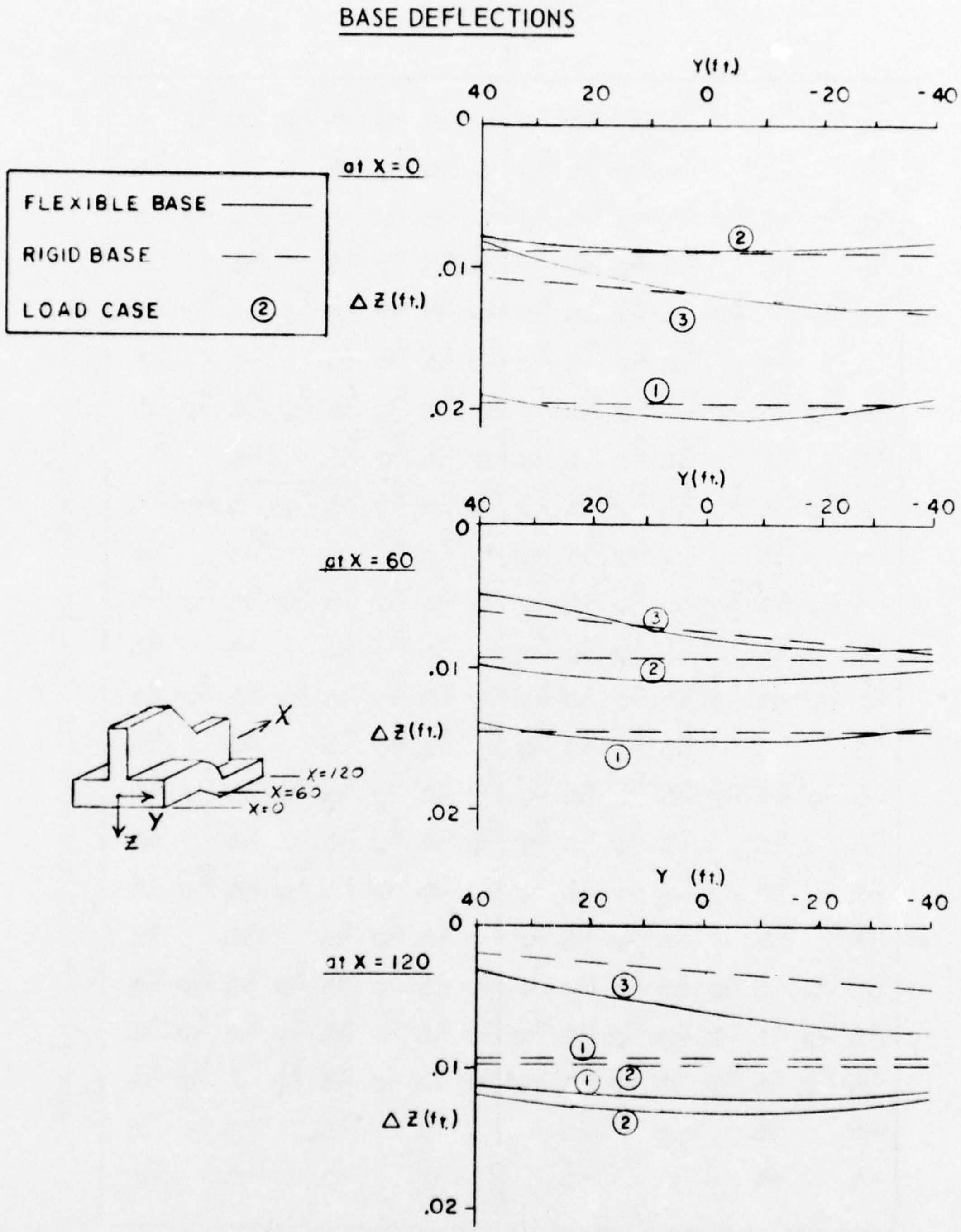


Figure A15. Comparison of results from the rigid and flexible base pile-group analysis - horizontal base deflection

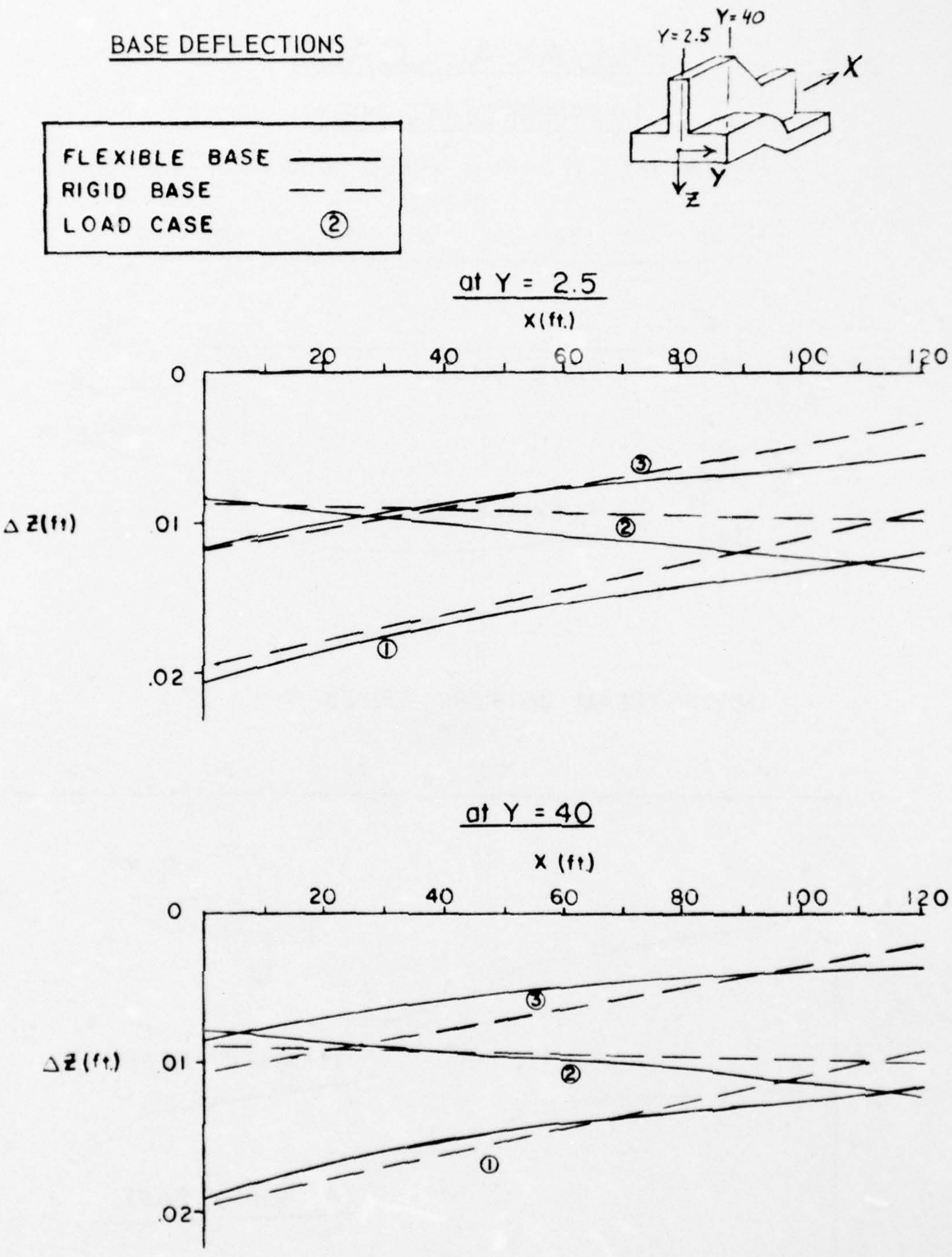


Figure A16. Comparison of results from the rigid and flexible base pile-group analysis - vertical base deflection

PILE AXIAL LOADS

INTERMEDIATE PIER

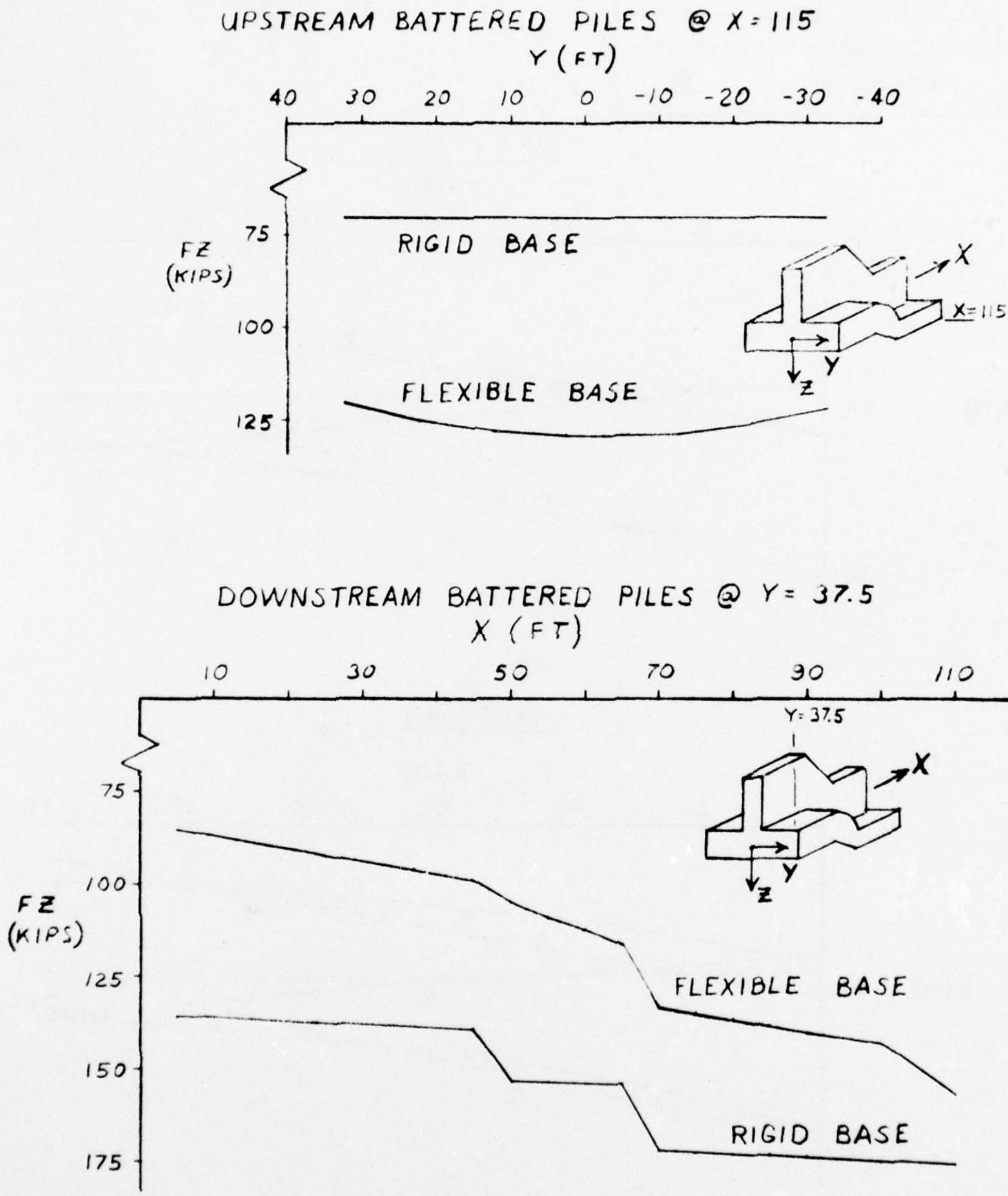


Figure A17. Comparison of results from the rigid and flexible base pile-group analysis - vertical base deflection

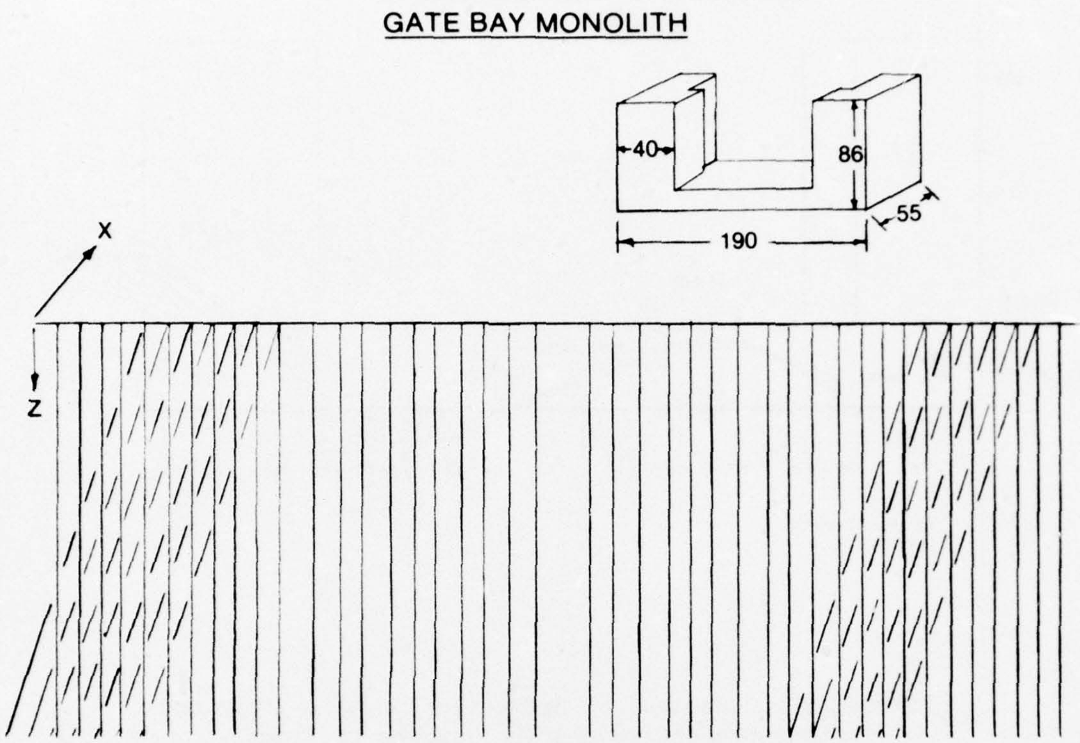


Figure A18. Pile layout for 2-dimensional rigid and flexible base analysis of a pile-founded lock gate monolith

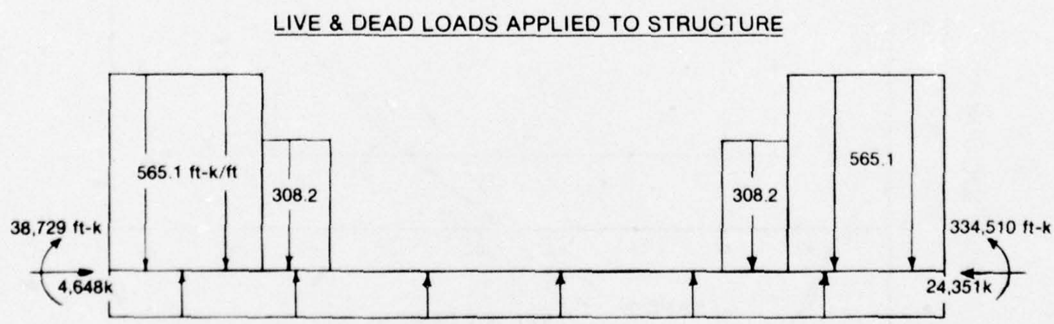


Figure A19. Loading conditions for 2-dimensional rigid and flexible base analysis of a pile-founded lock gate monolith

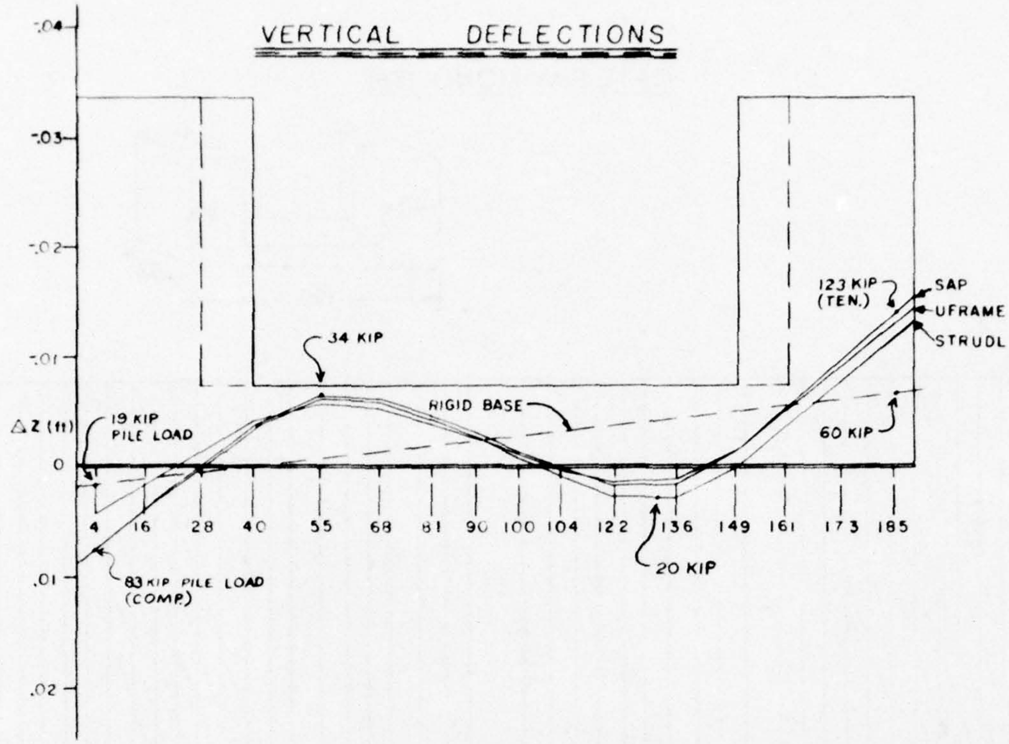


Figure A20. Results for deflection of pile-group and structure analysis using various programs

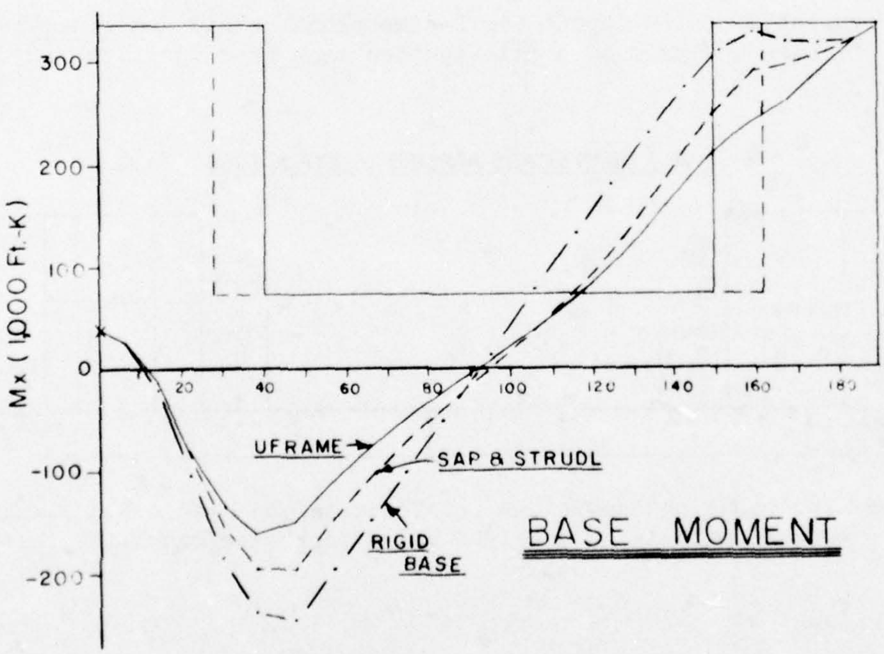


Figure A21. Results for base moments of pile-group and structure analysis using various programs

affects pile forces. The conclusion for this structure is that while a rigid base solution may be sufficient for determining base moments, a flexible base solution is required for determining pile forces accurately.

c. Pump station founded on vertical H-piling - A pump station was modeled with SAP PILE using plate and pile elements. There was some question on this job about the flexibility of the base as it affects the distribution of loads to the piles under the wall and base slab (Figures A22-A23). This finite element grid consisted of 163 nodes, 142 plate elements, and 118 pile elements (Figure A24). Results of the comparison between the rigid and flexible base assumption are shown in Figure A25. A conclusion which may be drawn from this analysis is that the rigid base analysis may be sufficient for this design, since the pile layout provided is conservative. However, the flexible base analysis indicates that more tension piles are required than the rigid base solution if this is the criterion to be used for placement of tension anchors.

Comments on Programs

a. 713F3A3840 (Rigid base solution) - Input is rather simple since only pile information is required; since the base is assumed to be rigid it need not be modeled. To obtain base forces, a second run must be made using another program which utilizes the output from 713F3A3840.

b. 713F3A3910 (UFRAME) - The input is simple since this is not a general program; it is only for two-dimensional analysis of pile-founded flexible base structures. Some limitations are that out-of-plane batter is ignored and that the pile is assumed to be connected to the centroid of the base, not the bottom of the base. An advantage of this program is that it is run on time sharing rather than batch.

c. STRUDL (MCAUTO) - Input is in engineering-type language, thus it is more understandable. However, STRUDL has no standard pile element. A pile must be represented by inputting stiffness matrix terms for the pile elements which the user has calculated, a serious limitation for the casual user. STRUDL is also the most expensive program to run. One advantage for this program is its ability to represent complex structural details (e.g., member eccentricities, member end joint sizes, curved members).

d. SAP PILE (WES batch) - This is a large general purpose program like STRUDL. SAP has useful automatic data generation features. The fact that it has a standard pile element in its library of elements makes it easier to model a structure than on STRUDL. SAP does have some limitations on loading input when multiple load cases are desired.

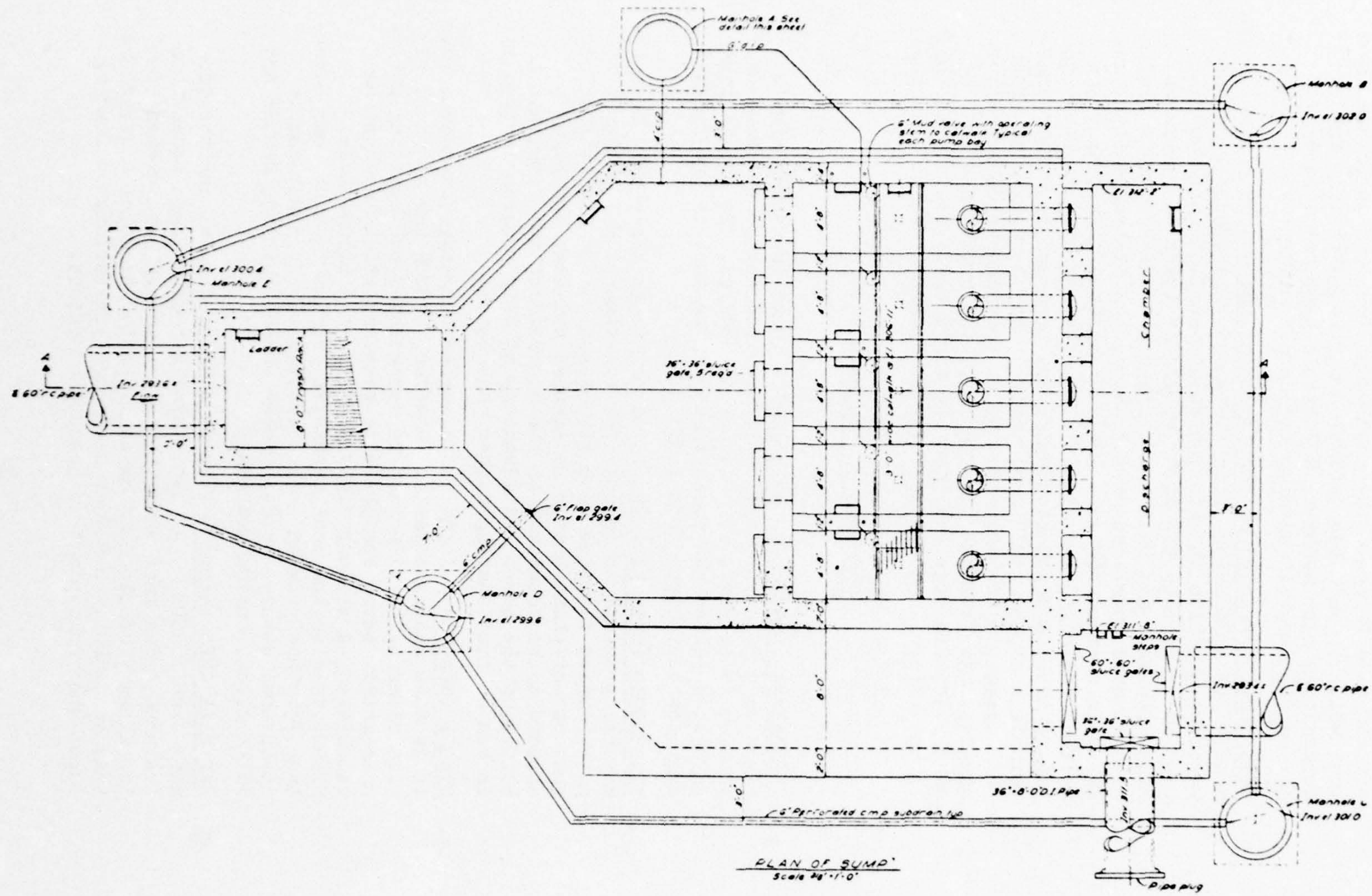


Figure A22. Plan of pile-founded pump station

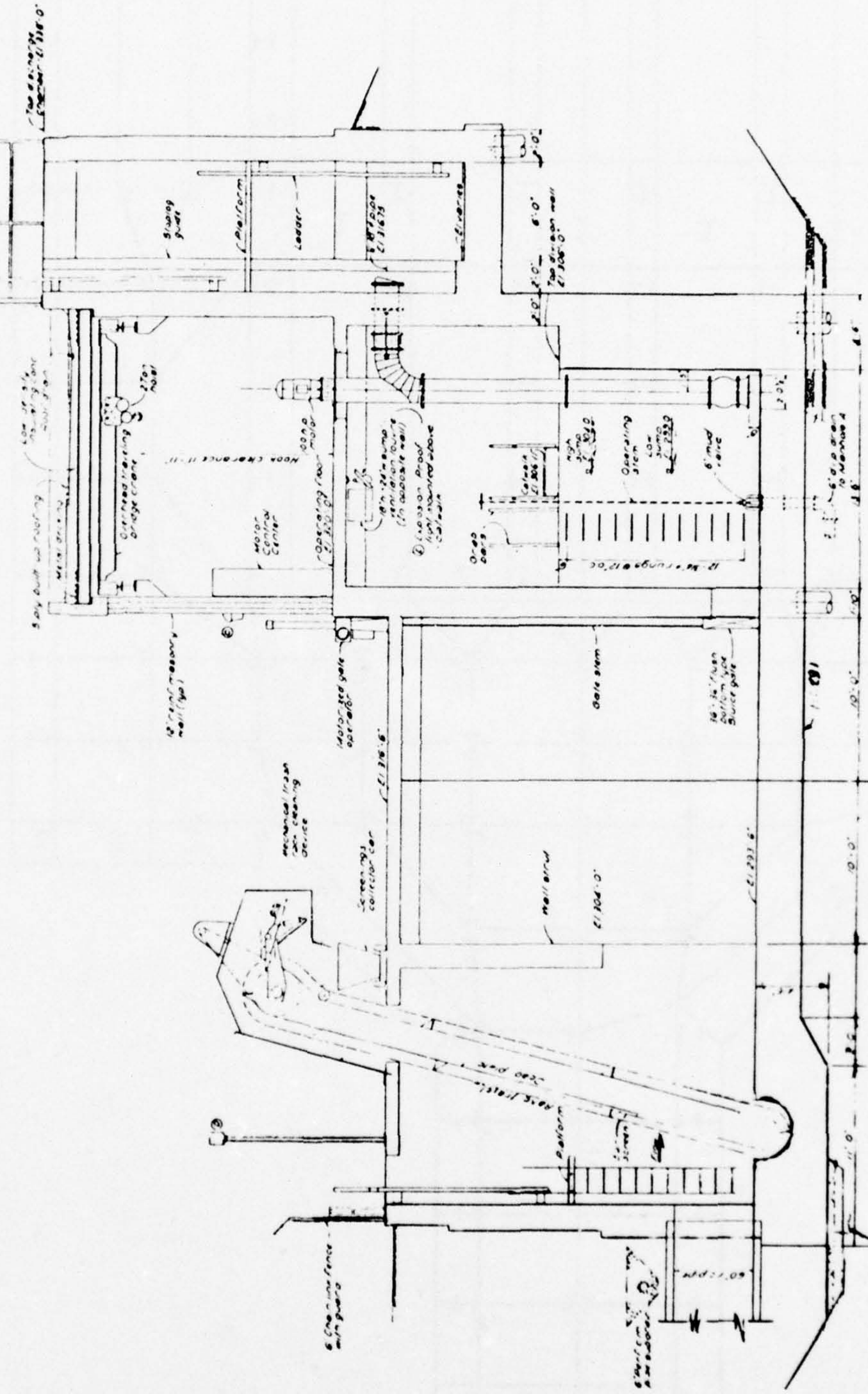


Figure A23. Cross section of pile-founded pump station

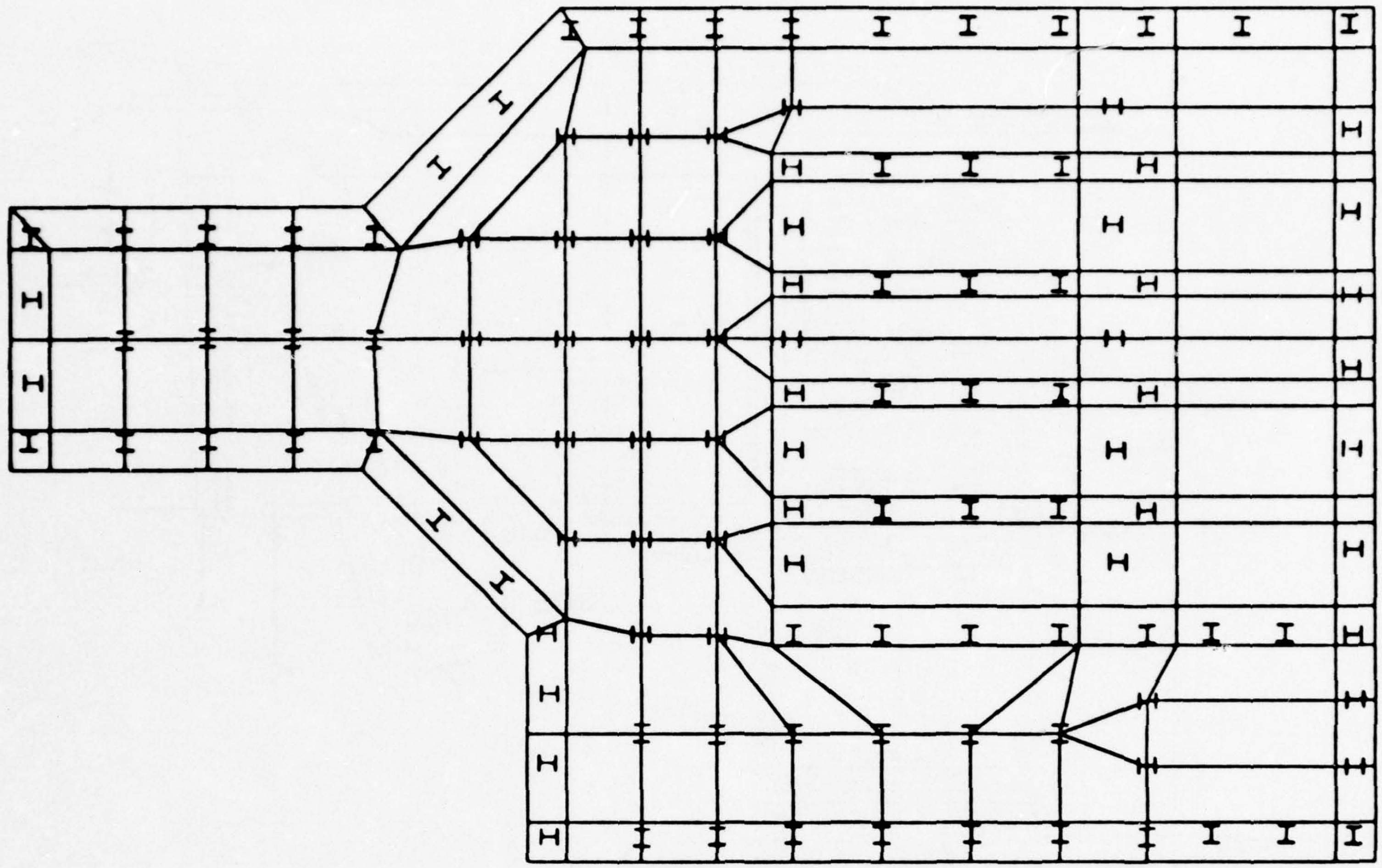


Figure A24. Finite element idealization of pile layout with plates for pile-founded pump station

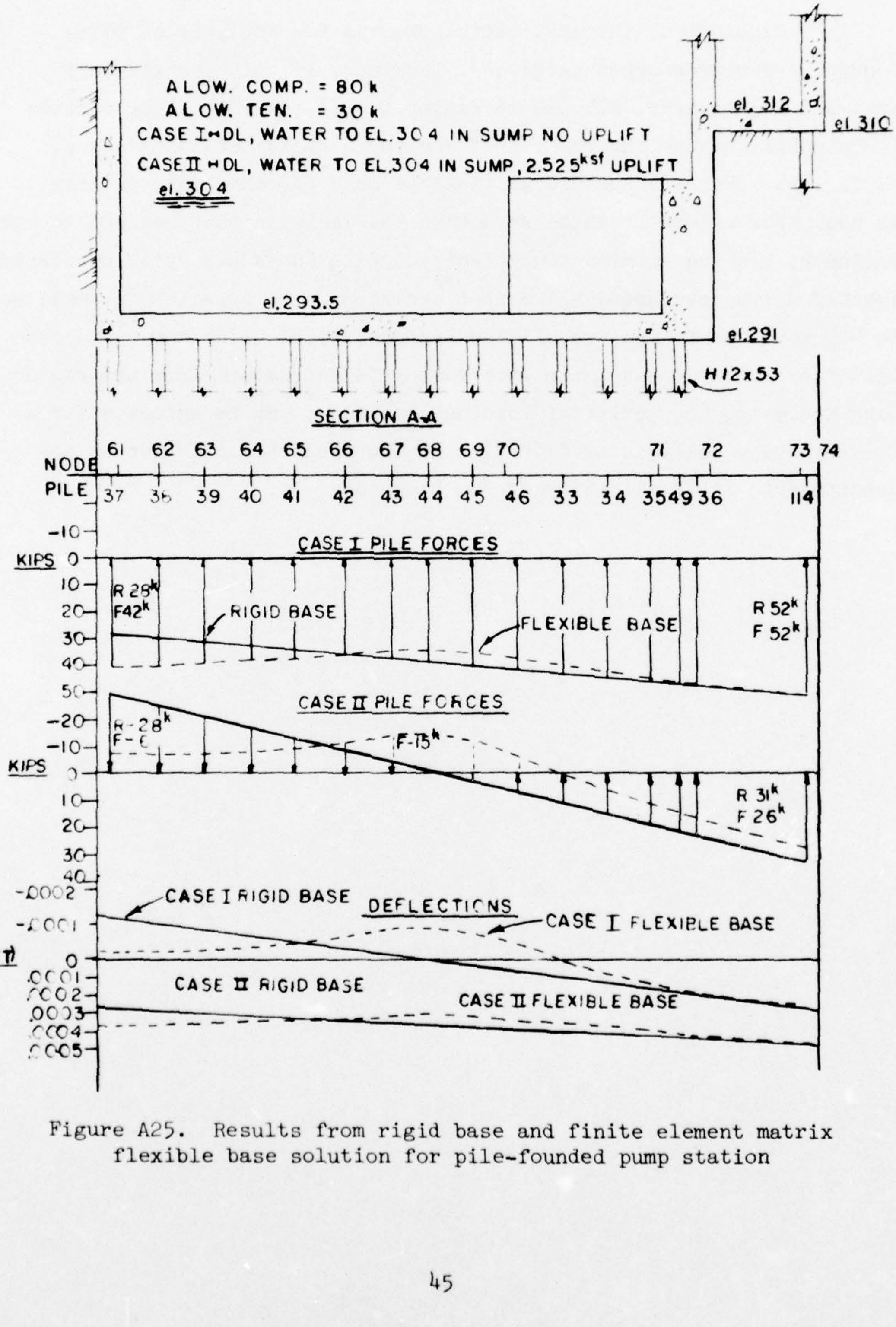


Figure A25. Results from rigid base and finite element matrix flexible base solution for pile-founded pump station

Observations

In conclusion, the most useful program for analysis of pile-founded structures seems to be SAP. However, if only a rigid base analysis is required, A713840 is easier to use than SAP in most cases. There still remains the problem of whether a rigid base analysis is sufficient. For a structure as flexible as a U-Frame lock, clearly it is not. For a less flexible structure the decision must be left to each engineer, bearing in mind that predicted pile loads may still be significantly different depending on that decision. The approach we would use in SLD would be to analyze all loading conditions using the rigid base solution. Once a reasonable pile layout is determined from the rigid base analysis, then critical loading conditions can be selected for a flexible base analysis to determine if the predicted pile forces are sensitive to the flexibility of the structure.

CELLULAR SHEET PILE STRUCTURES

by

Thurman Gaddie*
Herman Gray**

Introduction

The first interlocking steel sheet piling was of the Lackawanna type and was produced in the United States in 1908. This piling was used in the first cellular cofferdam, which was constructed in Buffalo, New York, in 1909. This cofferdam, of the diaphragm type but with all straight walls, served its intended purpose even though large deformations in the cells caused alarm. A circular-type cellular cofferdam was constructed the following year in the harbor of Havana, Cuba, in connection with raising the battleship Maine. The soft clay foundation and the use of clay fill material led to excessive cell deflections which required the placement of a stability berm and the eventual strutting of the cells. Despite these early difficulties, the potential of cellular structures was recognized and the deficiencies noted were corrected in subsequent designs.

Stability considerations for early cellular structures were quite simple. Sliding and overturning of an equivalent rectangular mass was the sole basis for stability until the late 1930's when the Tennessee Valley Authority (TVA) began its experiments involving the cell fill. Following these experiments, Terzaghi,[1] in 1945, first published his paper on vertical shear within the cell fill. Improvements in steel sheet piling have been made over the years permitting larger and higher cellular structures to be built. High interlock strength piling and compatible extruded wye connector piles are now available which will permit even larger and higher cellular structures to be built. Studies by Terzaghi,[1] Cummings,[2] Hansen,[3] and others have produced design

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methods which will assure safe designs for these larger cellular structures.

While computer programs have been written to perform some of the analyses required, more comprehensive design programs are still needed. The criteria for evaluating existing programs and for the preparation of future programs will be presented in the following paragraphs. The deficiencies in cellular structures which most commonly result in failure will also be discussed so that design criteria presented can be placed in proper context.

Cellular Structures--General Types and Usage

General types

Cellular sheet pile structures are built in numerous configurations. The most common of these are the circular cell, diaphragm, and cloverleaf configurations. The circular cell type is preferred where river currents must be contended with. The individual cells can be filled, thus providing an independently stable structure as work progresses on the remaining cells and arcs. Completed cells with arcs can be used as a work platform for construction of subsequent cells. On the other hand, the diaphragm-type cells represent an economical use of sheet piling for moderate height structures and can be used to advantage in still water. The cloverleaf configuration is often used where large cells are required for stability and where there is insufficient room for stability berms.

Uses

Cellular sheet pile structures are versatile in size and configuration and are used in a variety of ways, the most common being for cofferdams. Cellular cofferdams (Figure 1) are used to best advantage in river work where the river is blocked in stages so as to restrict riverflow as little as possible. The cells are resistant to scour when driven to rock or otherwise protected. Among the uses for permanent structures are: (a) fixed crest dams and weirs (Figure 2), (b) navigation lock walls (Figure 3), (c) mooring cells (Figure 4), (d) retaining

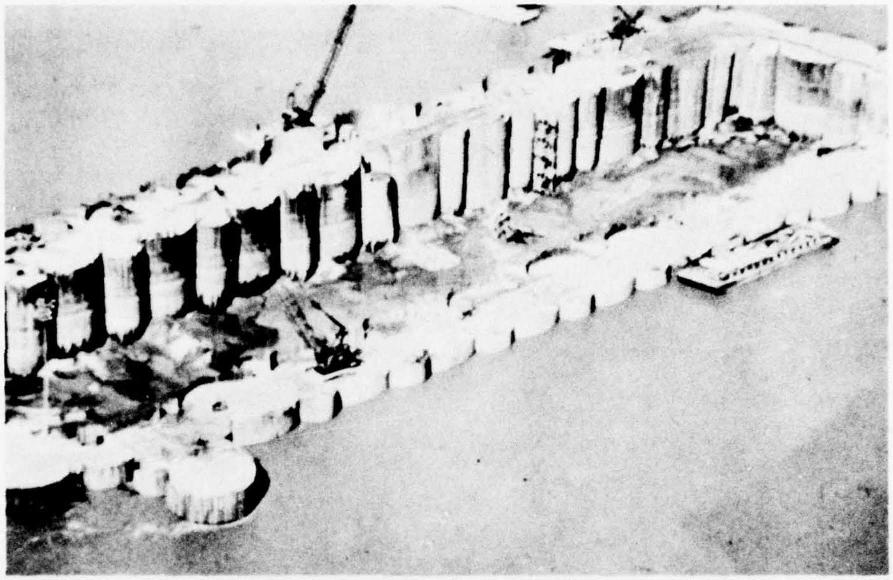

Figure 1. Cellular cofferdam Uniontown Dam Project, Ohio River, Indiana and Kentucky

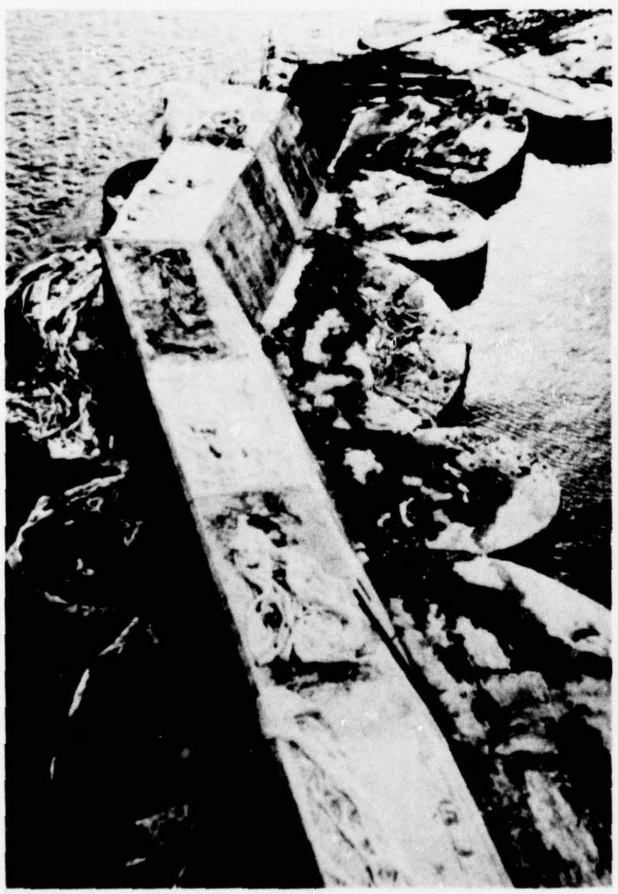

Figure 2. Fixed weir, Newburgh Dam Project, Ohio River, Indiana and Kentucky

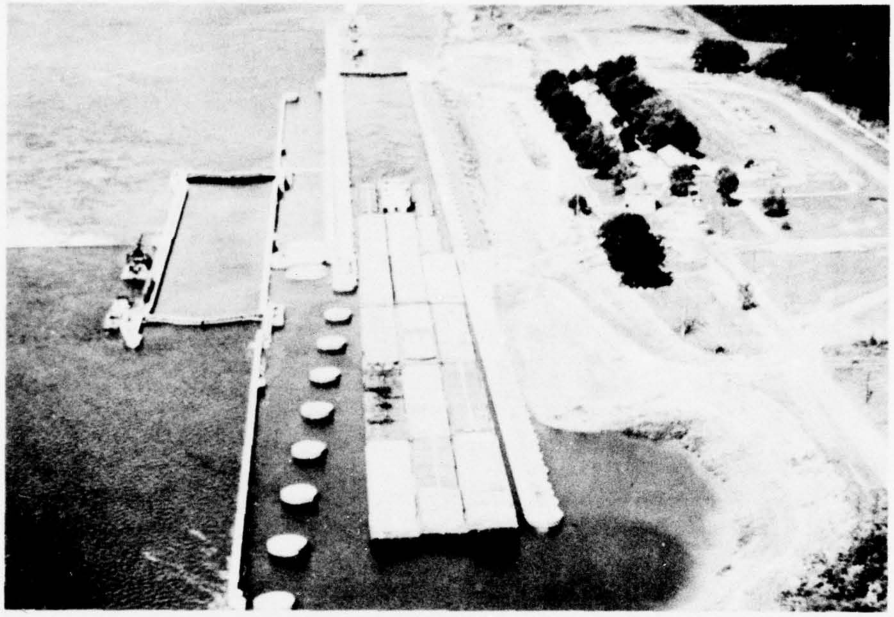

Figure 3. Navigation lock walls

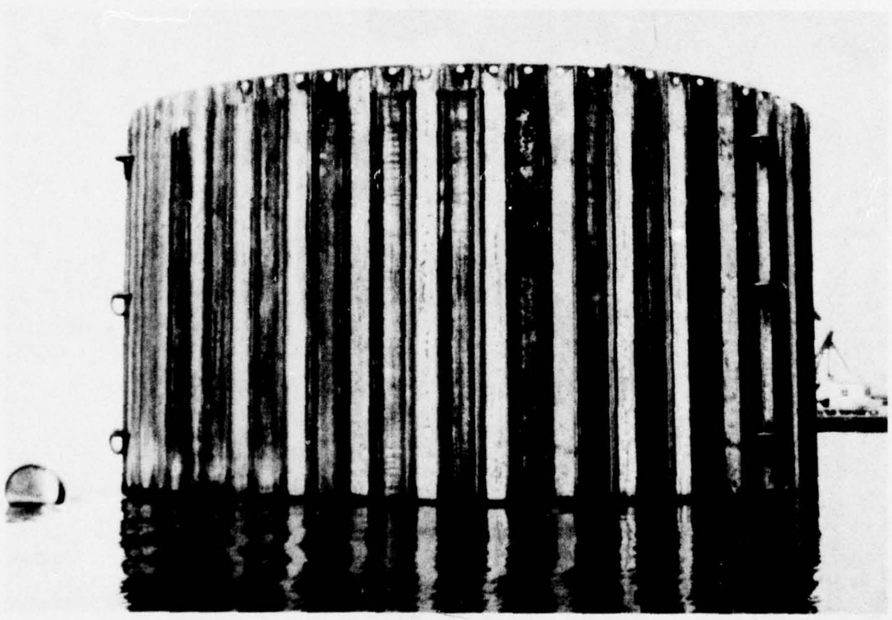

Figure 4. Mooring cells

walls (Figure 5), and (e) substructures for concrete gravity superstructures (Figure 6).

All of these structures can be built in the wet, thus eliminating the need for dewatering. When used as substructures, the cells can be relied upon to support moderate loads from concrete superstructures. Varying designs have been used to support the concrete loads, either on the fill or on the piling, by providing appropriate connections between the concrete and piling. In the case of concrete guard walls for navigation locks, bearing piles have been driven within the cells to support the load. Where the danger of rupture from barge impact exists, the cells may be filled with tremie concrete. Where settlement of the cell fill would pose a problem, material used for the cell fill and the means of consolidating the fill must be taken into account during design.

Design Philosophy

The design of cellular structures can be reduced to three major areas of consideration. These are:

- a. Design of the sheet piling and connectors to resist all internally applied loads from the cell fill (including hydrostatic forces).
- b. Design of the cells to be stable under all loading conditions, considering all possible modes of failure.
- c. Design for the control of the quantity of seepage, seepage forces, and erosive currents.

Design of Sheet Piling

The dimensions and configuration of sheet pile cells are normally limited to those which will result in reasonable stresses in the sheet piling and connector piles. The procedures followed in computing piling stresses vary widely with the designer and a decision as to what represents a reasonable stress involves a great deal of engineering judgment. Assumptions regarding earth pressures within the cells have been found to vary from the Rankine active state of stress to the Krynine[4] state of

Figure 5. Retaining walls

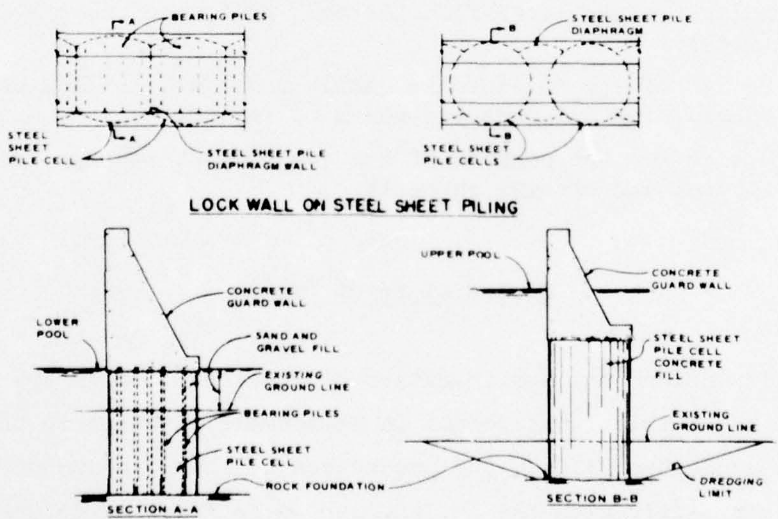

Figure 6. Substructures for concrete gravity superstructures

stress, which is nearly twice that of the active state. Interlock stresses vary with the depth of fill in the cells but are affected by the restraint to cell expansion offered by overburden or bedrock into which the piling is driven. The elevation at which maximum interlock stress will be computed varies from the base of cells to an elevation at which the designer estimates that maximum cell expansion will occur. While the allowable interlock stress is commonly taken as one-half of the guaranteed ultimate interlock strength, the more discerning designer may choose to consider such factors as: (a) the condition of the piling, (b) the conditions under which the cells are constructed, (c) the conditions under which critical loads are applied, and (d) the control (or lack of control) the designer may have during construction of the cells.

The maximum interlock stress normally occurs in the connector piling where hoop tension in the cells is combined with pull from the connecting arcs. This maximum interlock tension, T_{max}, for circular type cells can be computed from the equation:

$$T_{max} = pL \sec \alpha$$

where:

p = the fill pressure at the elevation under consideration

L = half of the spacing between cells

α = the angle between center line of cells and a line from the center of a cell to the connector pile in question

Similar analysis must be performed for diaphragm and cloverleaf-type cellular structures.

The single most important consideration in the design of the cells is the saturation level within the cell fill. Full saturation commonly occurs during cell filling operations. This can be a result of the means employed to fill the cells or due to sluicing operations performed to consolidate the cell fill. Near full saturation may also occur under other conditions, particularly if precautions are not taken to prevent such occurrences. For these reasons, the assumption of a fully saturated cell fill condition is normal practice. A notable exception of

this practice was in the design of a cofferdam for Smithland Dam on the Ohio River. Cells for this cofferdam are approximately 64 ft in diameter and will stand 41 ft above normal river level and 103 ft above the predredged riverbed. Stability berms will be provided. High interlock strength piling would be ideally suited to these cells; however, its availability could not be guaranteed at the time of design. The cofferdam was therefore designed to use conventional piling and free-draining stone fill. A saturation level nearly equal to the level of the river during fill placement was thus assured. Weep holes in the inboard sheet piling will maintain the cell saturation level near the level of berm saturation within the dewatered cofferdam.

Interlock stress determinations involve a number of loading conditions to describe the various stages of construction and operation and to consider the effects of varying design assumptions. While the maximum interlock stress may likely occur during the cell filling, other cases will be of interest to the designer. If the critical interlock stress occurs during filling and the interlock stresses are low under all other conditions, the designer may be inclined to allow a slightly lower factor of safety for the construction condition. This would assume that the filling condition represented a test loading and that failure could not occur so long as the test loads were not exceeded.

Limiting the computed interlock stresses alone will not assure the integrity of the cells. The critical link is often found to be the connector piling. Stresses in these members must be investigated for the same conditions considered for the sheet piling. Aside from the weaknesses of the connector piles themselves, their effect on adjacent sheet piling must be considered in arriving at a safe cell design. As studies by Dismuke[5] indicate, substantial stress in adjacent piling is induced by the pull on the outstanding leg of connector tees and wyes. If it were not for plastic deformation in these pilings, the stresses would be intolerable. While plastic deformation does occur, it is prudent that the pull on the outstanding leg be limited by designing the arcs with as small a radius as practical. Wyes are to be preferred over tees as the radial component of the pull on the outstanding leg is less for arcs of

equal radius. Welded tees in particular have been associated with numerous failures and should be avoided.

Finally, in regard to design of the sheet piling, an important consideration is that of the problems associated with construction. A recent analysis[6] of some 35 cofferdam failures indicates that while "splits" in cells are the most common form of distress, these "splits" can rarely be attributed to improper analysis. Instead, the major cause was found to be a result of using defective piling and simply driving the piling out of interlock. These causes can be eliminated where the designer is cognizant of the problem and prepares his design accordingly. The single most important design consideration would be to limit pile driving to 30 ft or less depending on the nature of the overburden.

In some cases this would require excavation prior to pile driving. Other design considerations would be to: (a) require setting of piles in connecting arcs adjacent to cells prior to filling cells so as to avoid driving these piles past a bulging cell; (b) require jetting on both sides of pile driven through dense overburden; (c) require that piling be driven so that no pile leads adjacent sheets by more than 5 ft; (d) require that cells over 80 ft high or cells placed in fast-flowing water use stiffened master piles; (e) require that piles driven in pairs be seated individually where they are driven to rock; and (f) require a diver inspection of completed cells to check for windows or other openings in cell.

Design of the Cells for Stability

Design theories relating to stability are quite divergent to the point of being contradictory. While many methods of analysis have been proposed, there are only four prominent methods of design in use today. These are the gravity wall analysis, Hansen's Method, Terzaghi's Method, and Cummings' Method. While all are widely used, no one is universally accepted to the exclusion of the others. Neither can it be said that all four methods yield approximately the same results. To go further, the results vary with the assumed fill and foundation properties and it

cannot be reasonably predicted which method will yield the most conservative design.

Gravity wall design

One leading consultant in the field of cofferdam design advocates a simple sliding and overturning analysis of an assumed equivalent "gravity block." While such an analysis is somewhat empirical, the procedure has been used with success for many years. This analysis is very sensitive to variations in the saturation levels within the cells. Using extreme assumptions, this method can yield either the largest or smallest cells of the four methods listed above. Factors of safety between 3.0 and 3.5 are considered adequate for overturning by designers who use this method. These factors of safety will maintain the resultant on or within the kern. Design factors of safety with respect to sliding are normally selected between 1.25 and 1.50.

Hansen's Method

Hansen also considers cellular structures to act as rigid bodies. His experiments and experiments by others tend to confirm his assumption that rupture planes occur near the base of cells rather than along planes within the cells. The Hansen Method represents a reasonable means of investigating sliding and tilting of cells founded in overburden. Stability is directly related to the engineering properties of the foundation and properly considers the saturation level within the cells as well as seepage forces beneath the cells. Sliding and rotation along various potential failure planes can be investigated by this method. No analysis of cells founded in overburden would be complete without such an investigation. The Hansen Method can also be applied to cells on rock; however, the most beneficial application is for cells in overburden.

Terzaghi's Method

Terzaghi's Method is a procedure for analyzing the tilting resistance of cellular structures in terms of the vertical shear resistance of the cell fill and the frictional resistance of the pile interlocks. Application of this method requires an assumption as to the state of stress within the cell fill. Terzaghi reasoned that the state of stress

would have to exceed the active pressure state if the apparent stability of existing marginal structures were to be explained. He therefore recommended that fill pressures between 120 percent and 150 percent of active earth pressures be assumed. Following the publication of Terzaghi's cell fill shear theory, Krynine presented an analysis indicating that the coefficient of lateral earth pressure should be taken as $K = \cos^2 \phi \div (2 - \cos^2 \phi)$. Krynine's value is commonly used today in evaluating the shear resistance of the cell fill. Active earth pressure, however, is most commonly used in computing stress in the interlocks. Also interlock stresses are commonly, but not always, assumed to vary from zero at the base of the cells to a maximum at the quarter height of the cells. Likewise, the frictional resistance in the interlocks is commonly assumed equal to zero at the base of the cells and to vary to a maximum at the quarter height of the cells. The assumption of zero interlock stress at the base of cells is considered reasonable where expansion of the cells is restricted at the base. The use of Krynine's earth pressure for cell fill shear and active earth pressure for interlock friction may be considered an inconsistency on the part of some designers. In any event, the variations in the manner in which vertical shear may be calculated tends to reduce the analysis to an empirical design. The saturation level within the cells has been found to have little effect on the calculated resistance of the cells to tilting. As the saturation level varies, changes in the cell fill shear resistance are offset by the changes in the interlock frictional resistance.

Cummings' Method

Cummings' Method relates cell tilting resistance to the shear resistance of the cell fill and to the resisting moment due to the frictional resistance of the pile interlocks. The shear in the cell fill is assumed to take place along horizontal planes. Active earth pressure within the cells is commonly used in computing the frictional resistance in the interlocks. As in Terzaghi's Method, the assumption regarding the distribution of interlock frictional resistance will vary with the designer. The saturation level in the cells has little influence on the results of the analysis. Frequently there may even be an apparent

increase in resistance to tilting as the saturation level in the cells is increased. This can be attributed to the significant increase in interlock frictional resistance resulting from the higher fill saturation level.

As inferred above, there is some variation in the application of Terzaghi's and Cummings' theories of cell stability. These variations lie primarily in the assumed distribution of interlock frictional resistance. The designer should be aware of these variations in assumptions and take note of their effect on the calculated stability factors of safety. Aside from the considerations of tilting, sliding, and overturning, there are other factors to be investigated which relate to stability. Cellular structures founded on rock are commonly investigated for adequate resistance to slippage between the cell fill and the outboard sheet piling. If such a slippage occurred, the outboard sheets would be lifted and fill would spill from the cells. Sliding along weak seams in the rock should also be investigated.

Cellular structures founded on overburden are commonly investigated for: (a) adequate foundation bearing capacity, (b) pullout of the outboard sheet piling, and (c) adequate penetration of the inboard sheet piling to resist further penetration due to the overturning moments applied to the cells.

Slippage between the cell fill and outboard sheet piling

This analysis assumes that the full overturning moment is applied to the sheet piles, which act as a rigid shell. The contact force between the piling and the cell fill is computed considering loads acting from outside the cell and effective lateral earth pressures within the cell. The coefficient of friction between granular cell fill and sheet piling will vary between 0.2 and 0.4 depending on the fill. A factor of safety of 1.25 against slippage at the cell fill-sheet piling interface is commonly accepted as adequate.

Sliding along weak seams in the rock

Failure within the rock foundation is a possibility that must be

considered in cofferdam design. Sedimentary rock formations frequently contain clay seams between competent rock strata. Sliding can take place along these seams where they are exposed in excavations or where they are intercepted by low angle fault planes. Adverse uplift conditions are commonly associated with such foundation conditions. The extent of open excavations in rock containing weak seams is an important consideration. If much of the rock is left intact, stability becomes a three-dimensional problem rather than two-dimensional, as most analyses assume. The geometric arrangement of the cells then becomes important if the stability is assumed to be a three-dimensional problem. The cells of a cofferdam for construction of a water intake for Cincinnati were arranged in a full circle in recognition of this.[7] A cofferdam of this type was recently designed in the Ohio River Division (ORD) (Figure 7) to be constructed on a questionable clay foundation. The support of each cell afforded by the adjacent cells was thus considered. Such practice, however, should be followed with caution.

<u>Foundation bearing capacity</u>

The bearing capacity of granular foundations is generally good

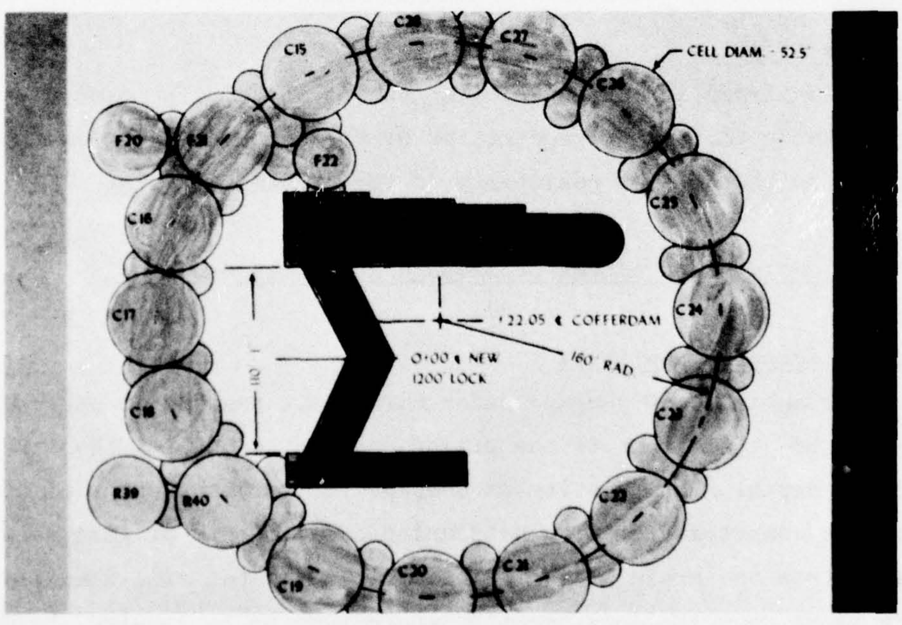

Figure 7. Circular cofferdam

where adequate penetration is provided and seepage forces are accounted for. The bearing capacity of cohesive foundations is dependent upon the consistency of the clays. The bearing capacities of both cohesive and noncohesive soils are commonly computed using the Terzaghi[8] curves and equations for continuous footings. These curves are based on an assumed failure condition whereby the footing punches into the ground causing passive blocks of earth to surface on each side of the footing. The failure planes assumed for the development of the Terzaghi curves and equations do not appear to be as realistic as those developed specifically for cellular structures by Brinch Hansen. Where bearing capacity is investigated, the Hansen Method of analysis is recommended.

Pullout of outboard sheet piling

This analysis is similar to that for slippage between the cell fill and the outboard sheet piling. The calculated overturning moments are applied to the sheet piles which are assumed to act as a rigid shell. Resistance to pullout is computed as the frictional forces or cohesive forces acting on the embedded length of piling. A factor of safety between 1.25 and 1.50 is generally considered adequate.

Penetration of the inboard sheet piling

This analysis in turn is similar to the above. The exception is that resistance to further penetration of the inboard piling is checked, as opposed to the pullout resistance of the outboard sheets.

Design for Control of Seepage and Erosive Currents

Control of seepage quantities

The quantities of seepage under cofferdams founded in pervious strata can be significant if the piling does not penetrate the strata to sufficient depth. The quantity of seepage for varying depths of penetration can be computed from flow nets using coefficients of permeability estimated from the grain size of the pervious strata. The quantity of seepage is an economic consideration where pumping is required to maintain a dry area.

Control of seepage forces

Seepage gradients can also be determined by flow nets. These gradients are required in the determination of effective stresses in the cell foundation and in the stability berms. Allowable bearing pressures, berm stability, and the computation of passive earth pressure are affected by the seepage forces.

Control of erosion

Erosion of overburden through which cellular sheet pile structures are driven is one of the most common sources of failure. The erosion results in loss of cell fill from either the base of cells or through splits which occurred below grade during driving. The possibility of erosion must be considered where it exists and the necessary provisions made during design to control or account for erosion.

Design Criteria

Corps of Engineers

The most recent criteria (Figure 8) is contained in the draft of EM 1110-2-2906, "Design of Pile Structures and Foundations," dated 16 November 1970.[9] The basic design considerations are as outlined below:

a. Maximum interlock tension is computed at the top of rock or, where overburden is present, at the top of overburden. Where interior berms are provided, interlock tension will also be computed at top of berm. Investigations involve all possible loading conditions including the cell fully saturated. Active earth pressures are used. A factor of safety of 2.0 is required.

b. Sliding at base of cells is investigated. No required factor of safety is stated.

c. The factor of safety against tilting is investigated by the Terzaghi Method. Krynine earth pressure coefficients are used for the computation of fill shear resistance. Interlock frictional resistance is computed using active earth pressure and a maximum interlock tension at the quarter height of cells is assumed. Factors of safety of 1.25 and 1.50 are required for temporary and permanent structures, respectively.

d. The factor of safety against tilting is also investigated by

	INTERLOCK STRESS	OVERTURNING OF GRAVITY BLOCK	TILTING BY TERZAGHI	TILTING BY CUMMINGS	ROTATION BY HANSON	SLIDING AT BASE OF CELLS	SLIDING IN ROCK	BEARING CAPACITY	SLIPPAGE BETWEEN CELL FILL & PILING	PULL-OUT RESIST OUTBOARD SHEET	PENETRATION INBOARD SHEET	SEEPAGE CONTROL
CORPS OF ENGINEERS	2.0		T 1.25 P 1.50	T 1.25 P 1.50		✓	✓	2.0		1.50		
TVA	2.0		T 1.25			T 1.25			T 1.25			
NAV FAC DM — 7	2.0		T 1.25 P 1.50			T 1.25 P 1.50		SAND 2.0 CLAY 2.5-3.0	1.30	1.50		✓
U S STEEL MANUAL	2.0		T 1.25 P 1.50	T 1.25 P 1.50	✓	T 1.25 P 1.50		SAND 2.0 CLAY 2.5-3.0	T 1.25 1.50	1.50	1.50	✓

✓ — CHECKED BUT NO FACTOR OF SAFETY SPECIFIED
T — TEMPORARY STRUCTURES
P — PERMANENT STRUCTURES

Figure 8. Design criteria - factors of safety

the Cummings Method. Required factors of safety and treatment of interlock friction are the same as for the Terzaghi Method.

e. Investigation of sliding below top of rock is required.

f. The bearing capacity under cells founded on sand is investigated. Ultimate bearing capacity is computed using the Terzaghi equation for continuous footings. A factor of 2.0 is required.

g. Cells founded on overburden are investigated for pullout resistance of the outboard sheets. A factor of safety of 1.5 is required.

Tennessee Valley Authority

TVA (Figure 8) uses Technical Monograph No. 75, Vol 1, "Cofferdams on Rock," dated December 1957,[10] as its guide in designing cofferdams. This publication considers only temporary structures founded on rock. Design considerations are as outlined below:

a. Maximum interlock tension is computed at the quarter height of cells. Active earth pressure within the cell is used in this determination. A factor of safety of 2.0 is required.

b. Sliding is investigated at top of rock. A factor of safety of 1.25 is required.

c. The factor of safety against tilting is investigated by the Terzaghi Method. Krynine earth pressure coefficients are used for the computation of fill shear resistance. Interlock frictional resistance is computed using active earth pressure with a maximum interlock tension at the quarter height of the cells. A factor of safety of 1.25 is required.

d. Slippage between the cell fill and sheet piling is investigated. A factor of safety of 1.25 is required.

Department of the Navy,
Naval Facilities Command

The Naval Facilities Command (Figure 8) uses NAVFAC DM-7, "Soil Mechanics, Foundations, and Earth Structures," dated March 1971,[11] for its guide in designing cellular structures. Guidance contained in that design memorandum is as follows:

a. Maximum interlock tension is computed at the quarter height of cells. A factor of safety of 2.0 is required and Krynine's earth pressure coefficient is used in computing interlock tension.

b. Sliding at base of cells is investigated. The required factors of safety are 1.25 for temporary structures and 1.50 for permanent structures.

c. The factor of safety against tilting is investigated by the Terzaghi Method. Krynine's earth pressure coefficients are used for the computation of fill shear resistance. Interlock frictional resistance is also computed using Krynine's earth pressure and assuming a maximum interlock tension at the quarter height of cells founded on rock and at the base of cells founded on overburden. Required factors of safety are 1.25 and 1.50 for temporary and permanent structures, respectively.

d. Slippage between the cell fill and sheet piling is investigated. A factor of safety of 1.30 is required for both temporary and permanent structures.

e. The bearing capacity under cells founded on overburden is investigated. Ultimate bearing capacity is computed using the Terzaghi equation for continuous footings. The required factors of safety, for both temporary and permanent structures, are 2.0 for cells founded on deep sand, 2.5 for cells on stiff to hard clay, and 3.0 for cells on soft to medium stiff clay.

f. The pullout resistance of the outboard sheet piling embedded in overburden is investigated for a required factor of safety of 1.5 for both temporary and permanent structures.

g. The pile penetration required for underseepage control is investigated using a flow net where cells are founded in overburden.

Bureau of Reclamation

The Bureau of Reclamation does not have any organization design criteria covering sheet pile structures. TVA criteria are commonly used by the Bureau of Reclamation.

United States Steel Corporation

The United States Steel Corporation (Figure 8) has an excellent design manual covering cellular sheet pile structures. This manual entitled "Steel Sheet Piling Design Manual," dated February 1974,[12] contains the following design criteria:

a. The maximum interlock stress is computed at the quarter height of cells using active earth pressure. A factor of safety of 2.0 is required.

b. Sliding is investigated at base of cells. Required factors of safety are 1.25 and 1.50 for temporary and permanent structures, respectively.

c. The factor of safety against tilting is investigated by the Terzaghi Method. Krynine's earth pressure coefficients are used for the computation of cell fill shear resistance. Frictional resistance in the interlocks is determined using active

earth pressure with the maximum occurring at the quarter height of the cells. Required factors of safety are 1.25 for temporary structures and 1.50 for permanent structures.

d. The factor of safety against tilting is also investigated by the Cummings Method. Required factors of safety and treatment of interlock friction are the same as for the Terzaghi Method. The cell fill, however, is assumed to be fully saturated for the cell fill shear computation. This is intended to simplify computations.

e. Slippage between the cell fill and sheet piling is investigated for a required factor of safety of 1.25 for temporary structures and 1.50 for permanent structures.

f. The bearing capacity of overburden supporting cells is investigated using the Terzaghi equation for continuous footings. The required factors of safety for both temporary and permanent structures are 2.0 for cells founded on sand and 2.50 to 3.0 for cells founded on clay.

g. The pullout resistance of outboard sheet piling embedded in overburden is investigated for a required factor of safety of 1.50 for both temporary and permanent structures.

h. The penetration of the inboard sheet piling is investigated for resistance to further penetration resulting from overturning moments applied to the cells. A factor of safety of 1.50 is required.

i. The pile penetration required for underseepage control is investigated using flow nets where cells are founded in overburden.

j. The possible failure by rotation of the cells founded in overburden is investigated by the Hansen Method.

Consultants

Generally, most consultants use the criteria of the particular agency by whom they are employed.

Computer Programs Available in the Corps*

Buffalo District

Cellular Sheet Pile Structure, Program No. 13J8F105.

a. Abstract. Designs cellular sheet pile walls by the Cummings Method to determine the equivalent width required for a factor of safety of 1.5 against tilting. Computes bearing pressure

* For capability of programs see Figure 9.

	INTERLOCK STRESS	OVERTURNING OF GRAVITY BLOCK	TILTING BY TERZAGHI	TILTING BY CUMMINGS	ROTATION BY HANSON	SLIDING AT BASE OF CELLS	SLIDING IN ROCK	BEARING CAPACITY	SLIPPAGE BETWEEN CELL FILL & PILING	PULL-OUT RESIST OUTBOARD SHEET	PENETRATION INBOARD SHEET	SEEPAGE CONTROL
NASHVILLE DISTRICT	A	X	A	A		A			A			
BUFFALO DISTRICT	A	X	X	A D	X	A		A				X
PITTSBURGH DISTRICT						A	A					

A — ANALYSIS
D — DESIGN
X — RECOMMENDED ADDITION

Figure 9. Programs - capability

at the top of overburden level. Determines the maximum cell radius that can be used without exceeding an allowable interlock stress of 8 kips per linear inch. Program can also be used to analyze a cellular wall of a given equivalent width to determine the factor of safety against tilting by the Cummings Method. Cell fill and overburden may be entered as a layered soil system. Loading may consist of a layered backfill with a layered soil system. Loading may consist of a layered backfill with a sloping surface, wave forces, differential hydrostatic loads, cell fill surcharge, and any specified additional overturning moment. The sliding factor at the base of cells is computed.

b. Documentation. The program was written by Elex M. Alter while with the Buffalo District. Documentation furnished by the Buffalo District is dated November 1966 with revisions dated October 1967 and June 1969. The documentation is good with regard to a description of the input data and information required to run the program. Lacking is a description of some analytical techniques used in the program. A set of manual calculations for a sample problem is included but does not illustrate techniques used for layered soil systems.

c. Evaluation of technical content. Consideration of the overburden and cell fill as a layered soil system is a practical consideration and a good use of the computer for the solution of a complex problem. This problem is frequently oversimplified when subjected to manual computations. The techniques used for the computation of active and passive earth pressures in layered soil systems are not described and cannot be evaluated. The use of the word "radius" in the documentation implies that interlock stresses are based upon a circular-type cellular wall. The analysis performed, however, is not described and it is not known how cell spacing and geometry are determined or accounted for. The program also appears to consider only horizontal saturation levels.

d. Recommendations. The documentation for this program should be improved so that design assumptions, analytical techniques, and program limitations can be readily evaluated by the user. Subject to the satisfactory revision of the documentation, the program should be placed in the Waterways Experiment Station (WES) library of computer programs. Consideration should be given to expanding the program to include analysis by the other commonly used methods. A possible modification of the program to improve flexibility would be optional use of sloping saturation levels.

Nashville District

Analysis of Circular Cofferdam and Mooring Cell Founded on Rock, Program No. 713-23-190.

a. Abstract. Analyzes a given circular steel sheet pile cofferdam or mooring cell founded on rock, using the methods presented in U. S. Steel's "Steel Sheet Piling Design Manual," dated February 1974, pages 68-77. One exception is that maximum interlock tension is computed using "Krynine's" earth pressure coefficient for the cell fill in lieu of active earth pressure. The program also investigates sliding at the base of cells, slipping between cell fill and sheet piling, vertical shear on the center line of cells and horizontal shear within the cell fill.

b. Documentation. The program was written by Walter Green and Randall Warren of the Nashville District. The program is used on the District's G-225 but is not formally documented. The design assumptions and analytical techniques, however, are well described in the referenced U. S. Steel Sheet Piling Design Manual. Messrs. Green and Warren have also prepared annotated input data sheets which allow anyone familiar with the design methods to use the program.

c. Evaluation of technical content. The analytical techniques used appear to be a reasonable application of Terzaghi's and Cummings' theories even though a number of simplifying assumptions are made. The following are assumptions which should be noted.

(1) The properties of any overburden within the cells are assumed to be the same as those of the cell fill.

(2) The angle of internal friction of the berm is assumed to be the same as that of the cell fill.

(3) Horizontal shear within the cell fill is computed assuming that the cell fill is fully saturated to the top of cell.

(4) The phreatic surface within the cell fill is assumed to be a straight line which originates at the pool level on the outboard sheet piling.

d. Recommendations. The program should be documented in accordance with ETL 1110-1-45, dated 9 February 1971, and should be placed in the program library maintained by the WES. Consideration should be given to revising the horizontal shear computations to use the sloping phreatic surface within the cell fill as described by the input data. Consideration should also be given to revising the input data to give the user more flexibility in selecting phreatic surfaces within the cell fill.

Pittsburgh District

Cofferdam Sliding Stability, Program No. 713-24-300.

a. Abstract. Analyzes the sliding stability of cellular sheet pile cofferdams founded on rock both at top of rock and at

planes of weakness below top of rock. Differing failure mechanisms are investigated including various block and wedge analyses and sliding along horizontal planes. The effects of berms and/or open excavations are considered. Uplift on the failure plane beneath the cells can be controlled by input data. Uplift on the failure plane beyond the limits of the cells is controlled by program assumptions. Factors of safety against sliding are computed. Pool elevation may be incremented automatically with prescribed changes in uplift so as to tabulate factors of safety for various pools.

b. Documentation. The program was written by Anton Krysa of the Pittsburgh District. The program is well documented and includes sample manual computations. The program can be used on a G-225 computer.

c. Evaluation of technical content. The theory of sliding along planes is simple and the application is straightforward. The input is variable so that forces acting on the failure planes can be controlled by the program user. The limiting assumptions of the program are well documented. The one limiting assumption which may be objectionable is that the failure plane for cross-bed shear through rock is set at $\alpha = (45° - \phi/2)$ with the horizontal. Where low angle fault planes are encountered, these should determine the failure plane to be investigated.

d. Recommendations. The program should be revised to allow the angle of cross-bed shear to be entered as input where it is set by geologic conditions. Subject to this revision, the program should be included in the library maintained by the WES.

New England Division

Cellular Cofferdam Design on a Rock Foundation.

a. Abstract. Analyzes a cellular sheet pile cofferdam for resistance to sliding and to tilting using the Terzaghi and Cummings Methods. The maximum interlock tension is checked at the top of the rock. The procedures are basically as outlined in EM 1110-2-2906 Draft, dated 16 November 1970. The cell loading is for water loads only and no berm is considered. Slippage between cell fill and sheet piling is not investigated.

b. Documentation. The design procedures are presented clearly and sample output is included. The program is for a Mathatron CS3 programmable calculator.

c. Evaluation of technical content. While limited in scope, the technical content of the program is good. The use of finite slices to evaluate horizontal shear with a sloping phreatic surface is considered exceptional for a program of this type.

d. Recommendations. While the New England Division program has its merits, conversion of the Mathatron statements to FORTRAN statements is not considered warranted.

Other governmental agencies

None of the major Federal agencies which would be involved in cofferdam design or related work, i.e., the TVA, the Bureau of Reclamation, or the Department of the Navy, Naval Facilities Engineering Command, use or have available any computer program for the design or analysis of cellular sheet pile structures.

Private industry

A random survey of several leading consulting engineering firms indicated that none of them used a computer program for the design or analysis of cellular sheet pile structures. United States Steel does have a program to determine the geometry of circular type cellular structures using 40-deg extruded wyes.

Need for New Programs

The programs available have been written by the individual offices to meet their needs as they saw them at the time. As the work in the originating offices varies, there is considerable diversity in the programs written. The programs are generally very good and have served their intended purpose well. An exchange and sharing of these programs would be beneficial to all involved. Improved design capability would be a direct result of this interchange.

In evaluating the need for new programs, it would be well to reexamine the analyses performed by the available programs. The Pittsburgh District program deals exclusively with sliding in rock. This is a unique problem worthy of the special program written to deal with the problem. Where such a problem exists, this program should be used. It is not recommended that this program be combined or used in part with any other program.

The Nashville District program deals exclusively with structures founded on rock. All of the analyses required are performed. The

program, however, lacks the versatility of loading offered by the Buffalo District program. The program also lacks some of the refinements in analytical techniques offered by the New England Division program. In short, with some modifications, the Nashville District program could be made into a suitable program for any cellular structure founded on rock.

While the Buffalo District program can be used for cellular structures in overburden, it does not perform all of the analyses normally required. With the addition of alternate methods of analyzing stability and determining the depth of pile penetration required, this program would be ideally suited to the design of cellular structures founded in overburden. Thus it would appear that with some modification, the existing programs available should satisfy the needs of the Corps of Engineers.

Conclusions

While the theories of analysis of cellular structures are quite divergent, they can still be used to establish the reasonable limits of good design. Where computers are used in the application of these theories, it is important that the programs be well documented. With some modifications, the Corps of Engineers has three computer programs which can be used as a sound basis for the design of cellular sheet pile structures.

Discussions on Paper

Four recommendations were made following presentations of the paper. These are discussed below.

a. Change the Pittsburgh District program to compute safety factors for sliding along inclined planes in rock. This refinement could and probably should be made to the program.

b. Revise programs to perform design functions as opposed to analysis. This is not considered warranted for the Pittsburgh District program. The Buffalo District program already contains this capability with respect to the Cummings Method. The suggestion is still worthwhile for the Nashville District

program. The program, however, should be flexible enough to allow the designer to select the method used for design and those used as check analyses.

c. Include program or programs as an appendix to EM 1110-2-2906. This would be appropriate.

d. Rename programs with names descriptive of what the programs do. While it is desirable to have appropriate names, the description of what the programs do can more appropriately be covered in the abstract, which is also generally brief.

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APPENDIX A: BIOGRAPHICAL SKETCHES OF AUTHORS

Mr. Billy H. James graduated in 1960 from the University of Arkansas with a B.S. in Civil Engineering and in 1970 earned an M.S. in the same field from the University of Illinois. He has worked as a structural engineer for the Corps in the Little Rock District, LMVD, and in Southwestern Division. He is a member of ASCE and is registered as a Professional Engineer in Arkansas.

Mr. Thomas J. Mudd graduated in 1961 with a B.S. in Civil Engineering from St. Louis University and in 1970 earned an M.S. in the same field from the University of Missouri at Rolla. He is currently serving as a supervisory structural engineer for the St. Louis District. He is a Registered Professional Engineer in the state of Missouri, and a member of ASCE and the Drafting and Design Technology Advisory Committee for the Meramec Community College. In 1969 he presented a report on the "Analysis of Pile Foundations" for the LMVD Structural Conference. He has done structural design work on the Meramec Basin Studies, Monsanto Pump Station, Shelbyville Reservoir Main Dam, Locks 27 West Wall Alterations and Guide Walls, Kaskaskia Lock and Dam, Clarence Cannon Main Dam and Reregulation Dam, the replacements for Locks and Dam No. 26, and the Teche-Vermilion Basins, La., Pump Station and Appurtenant Features.

Mr. Herman Gray served in the Army Air Corps for 3 yr in World War II, two of these years in England, European Theater of Operations. After WW II he attended Vanderbilt University and graduated in 1949 with a B.E. degree in Civil Engineering. He has worked for Nashville District since 1949, holding positions as civil engineer, structural engineer, and Chief, Civil & Structural Section, Design Branch. He is presently serving as Chief, Design Branch. Mr. Gray is a registered Professional Engineer in the State of Tennessee and is a member of the ASCE, Society of Value Engineers, Society of American Military Engineers, and the Engineers Association of Nashville, Tennessee.

In accordance with ER 70-2-3, paragraph 6c(1)(b), dated 15 February 1973, a facsimile catalog card in Library of Congress format is reproduced below.

Corps-Wide Conference on Computer-Aided Design in Structural Engineering, New Orleans, La., 1975.
  [Proceedings ... held in New Orleans, Louisiana, 22-26 September 1975] Vicksburg, Miss., Automatic Data Processing Center, U. S. Army Engineer Waterways Experiment Station, 1976-
  12 v. illus. 27 cm.
  Contents.-v.1. Management report.-v.2. List of computer programs for CADSE.-v.3. Invited speeches and technical presentations.-v.4. Division presentations.-v.5. State-of-the-Corps-Art (SOCA) reports on gravity monoliths, U-frame locks, and channels.-v.6. SOCA reports on gates, stoplogs, and trashracks.-v.7. SOCA reports on single- and multiple-cell conduits and tunnels.-v.8. SOCA reports on pile foundations and sheet pile cells.-v.9. SOCA reports on sheet pile walls and T-walls.-v.10. SOCA reports on stiffness methods, frames, and military construction.-v.11. SOCA reports on earthquake and dynamic analyses.-v.12. Interactive graphics, SEARCH and CORPS systems.

(Continued on next card)

Corps-Wide Conference on Computer-Aided Design in Structural Engineering, New Orleans, La., 1975.
  [Proceedings ...] 1976-
  (Card 2)

  1. Computer-aided design -- Congresses. 2. Design -- Congresses. 3. Structural engineering -- Congresses. I. U. S. Army. Corps of Engineers. II. U. S. Waterways Experiment Station, Vicksburg, Miss.
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